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EFFECT OF LOAD PATH AND FAILURE MODES ON SEISMIC RESPONSE OF REGULAR BRIDGES

by

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ABSTRACT

Bridges are essential infrastructure constituents that have been studied for centuries. Typically, seismic bridge design and assessment utilize simplified modeling and analysis techniques based on one-dimensional spine elements and zero-length springs/hinges. The geometry of the elements and calibration of parameters are based on assumptions for the lateral load path and failure modes, e.g., sacrificial backwall and shear keys, neglecting wing walls, and strength based on backfill alone. These assumptions have led to observations of underestimated resistance, overestimated displacement demands, and unpredicted damage and failure mode. The focus of the study is on ordinary standard bridges with continuous reinforced concrete box girder superstructures and seat-type abutments.

A bridge component calibration study was conducted first using simplified (spine models with 1D elements and springs) and three-dimensional nonlinear continuum finite element models (FEM). Model responses were compared with experimental results to identify the drawbacks in the simplified models and verify the adequacy of the material nonlinearities and analysis procedures. The components include a T-girder, abutment backfill, abutment shear key, elastomeric bearing pad, and a bridge pier. Results show the simplified models do not capture damage propagation and failure mode in the shear key case, nonlinear behaviors in beams with high aspect ratios (or deep beam action), and underestimate the strength and overestimate the stiffness for the backfill case.

The component models (both simplified and continuum) were then used in studying the nonlinear static behaviors of key bridge lateral-load resisting substructures, namely abutments and bents. For the abutment subsystem, cases with and without backfill and several back wall construction joint configurations for the longitudinal direction, with monolithic shear key and shear key with construction joint for the transverse direction, and boundary conditions in the transverse direction
were considered. Abutment subsystem results showed simplified models underestimate the resistance by 10-60%, neglect back wall and wing wall structural contributions, and localize damage in the back fill relative to the continuum models. For the bent subsystem, a full bridge system that considers material nonlinearity and damage in the bent segment only was adopted to determine the effect of the finite bent cap or superstructure-to-column connection. Inelastic behavior and damage was included in the columns, bent cap, and a superstructure segment with a length that correspond to the dead load moment inflection point. The other superstructure segments and the pile cap were modeled as elastic. Bent subsystem results showed simplified models overestimate the stiffness, induce excessive flexibility and deformation in the cap beam, and overestimate columns’ deformations.

Due to the differences observed in the abutment subsystem, and the potential impact of the abutment behavior on the seismic response of the whole bridge system, dynamic studies on the bridge system were conducted using four abutment parameters: abutment stiffness and strength in each of the longitudinal and transverse directions. Two models were developed to conduct nonlinear time history analysis: an equivalent single-degree-of-freedom (SDOF) model for each of the longitudinal and transverse directions, and a 3D spine bridge model. Constant ductility analyses were conducted using the SDOF systems, while standard probabilistic seismic demand analysis was used on the spine systems.

Results revealed that, besides the columns yielding, the abutment has an early and significant contribution to the behavior. The SDOF system results showed that increasing the abutment stiffness or strength reduces the system displacement demand and increases the system forces. The consequence of such increase in the forces is mobilizing significant amount of force in the abutments, causing inelastic response. The full bridge study also confirmed the SDOF results and showed that the abutment forces are more than 200% of the columns forces that would result in the same aftereffect observed in the SDOF system.
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CHAPTER 1: INTRODUCTION

Bridges are among the earliest and most important structures that helped to develop societies. As a result of this importance, bridges have been studied for centuries, and researchers have used several techniques trying to understand the behavior of bridges and predict a realistic bridge failure modes and damage assumptions. At the beginning of the twenty-first Century, bridge seismic design and analysis became indispensable especially after important events like the 1985 Chile earthquake, the 1994 North-ridge earthquake, and the 2014 Iquique earthquake. Earthquakes are characterized by their ability to identify the structural weaknesses and concentrate the damage at these locations. Besides their sensitivity to several problem areas, bridges have little or no redundancy in the structural system, and failure of a bridge element or the connection between these elements will likely lead to the bridge collapse Priestley et al. [105]. Considerations above make bridges’ seismic design and analysis critical and essential processes.

Typically, a bridge’s design and analyses depend on one of two types of assumptions: (1) load path assumptions and (2) damage and/or failure mode assumptions. Load path (force transfer mechanism) can be defined as the path that the load uses to transfer from the superstructure through the bridge parts to the ground or the path of earthquake excitation from the ground through the bridge parts. Failure modes and damage sequence can be described as the specified bridge potential failure modes and fusing sequence for each bridge part and their effect on the other bridge components and the global bridge behavior. These assumptions affect the bridge components’ design parameters, cross sections, and the adopted components’ strength and capacity.

Despite the significant efforts that have been done by researchers to predict and understand bridges’ behavior and the fact that bridges can be considered as simple structural systems, seismic response of bridges is still not fully understood yet. Significant differences were observed between the
predicted and observed damage and failure modes of bridges. Several reconnaissance reports indicated unexpected local and global bridge damage such as superstructure unseating [19, 38, 67], shear key and back wall failure [67, 122, 63, 145], excessive abutment foundation displacement [60, 122], excessive deformation of embankment and back fill soil [67], superstructure displacement and rotation [79, 44], and column bent failure [90].

Such consequences could be attributed to many reasons. The first reason is the use of simplified modeling and analyses procedures, which utilize one dimensional (1D) elements and simple mathematical representation, that do not usually capture the correct load path and failure modes. The assumptions and theoretical resistance mechanisms used in these 1D elements formulations limit their ability to predict the correct component behavior. The second is the design codes’ hypotheses related to the load path and components’ failure modes adopted in these procedures. For instance, Caltrans Seismic Design Criteria (SDC) 2013 categorizes bridges’ components response into sacrificial, critical, and capacity designed components which do not reflect the actual components’ behavior and induce different bridge response. Finally, the third reason is the neglect of several resistance resources, such as the contributions of several bridge components and phenomena such as the friction which also results in inaccurate bridge response predictions. Above consequences necessitate further assessment for the effect of using simplified modeling and analyses procedures on bridges’ response and the appropriate load path, failure modes, and damage sequence that should be considered in these procedures. The outcome will provide better understanding of the consequences of such considerations on the analysis and design results, which in turn leads to better estimation of the bridge behavior and its components’ seismic demand and capacity prediction.
1.1 Bridges Seismic Design Criteria

The main purpose of bridge seismic design is to save lives through the prevention of bridge collapses during earthquake events and allow temporary use of bridges by emergency vehicles after such events. Each design code has its own assumptions for the load path, failure mode and mechanism, and philosophy regarding each component response, which are utilized to specify bridges’ design perspectives and parameters. This section illustrate several design codes hypothesis regarding bridges’ abutment and column bent components and the adopted load path and failure modes for these systems.

1.1.1 Design Criteria for Bridge Abutments

1.1.1.1 Caltrans SDC and AASHTO Specifications

Several classifications can be used to characterize bridges’ components such as classifying the components into superstructure, substructure, and foundation components or subdividing bridges into subsystems such as abutment subsystem, superstructure subsystem, and column bent subsystem. Furthermore, design codes commonly characterize bridges’ components according to their predicted response and the hypotheses used in their design.

Caltrans SDC 2013 categorizes some of the components as the capacity protected components, which, according to the design criteria, should be designed to sustain the elastic behavior until the columns reach their overstrength capacity. Capacity protected components are footings, bent cap beams, type II shafts, joints, and superstructure which must be designed to maintain a ductile failure mode. To illustrate, SDC 2013 states that the nominal superstructure longitudinal capacity and the nominal bent cap transverse capacity must be sufficient to ensure that columns’ elastic
limit should have been well exceeded before the superstructure and the bent cap reach their nominal strength. Other cases for capacity protected components are the stem wall horizontal reinforcement that should be calculated to resist the shear key forces elastically and seat-type abutments that are designed to resist the transverse loads elastically. Another category is the ductile members or the seismic critical members which are the members that are designed to deform inelastically for several cycles without significant degradation in the strength or stiffness under demands generated by the Design Seismic Hazard [20]. These members include pier walls, Type I shafts, Type II shafts in soft soil, piles/shafts groups, pile or pile extensions in slab bridges, and all of the bridge columns. The design criteria allow these members to experience damage in seismic events without causing any structural collapse or loss of structural global response. The last category is the sacrificial components, such as shear keys and abutment back walls, which are usually designed to break off and transfer the loads between the other components instead of resisting these loads.

The load path and failure mode assumptions have a significant effect on the seismic design of bridges. There are many examples of load path and failure mode dependent design and for seated type abutments, in specific. AASHTO publications [2, 3, 4] in addition to Caltrans [20] assume that seismic critical members such as piers should be designed to deform inelastically with plastic hinges at their ends and considered the only resisting components to the lateral seismic load with minimal or no contribution of the abutments. These references also recommend that abutments’ back walls should be designed to break off and transfer the loads from the superstructure to the embankments that are considered as the main resisting elements to the superstructure longitudinal movement. Shear keys should dissipate the energy resulted from superstructure impact, protect the superstructure from unseating, and prevent transferring the lateral shear forces to the abutments and the foundation systems. In fact, Caltrans [20] requires using a smooth construction joint at the shear key interface between the stem wall and back wall in order to concentrate the shear resistance on the reinforcements and isolate the shear key from the other abutment components.
As a conclusion, understanding each bridge components’ local behavior plays an important role in choosing the proper load path and failure modes assumptions that will be considered in the simplified models that are commonly adopted in bridges’ design and assessment processes.

Figure 1.1: Longitudinal load path and failure mode of bridge’s abutment according to Caltrans [20].

Figure 1.2: Transverse load path and failure mode of bridge’s abutment according to Caltrans [20].
Japan Road Association (JAR) specifications [59] and Eurocode 8 (EC8) [34, 35, 36] with an inelastic behavior point of view, consider the shear keys as critical structural components that should maintain elastic behavior and backwall as a non-critical component with a non sacrificial role. In a point of fact, both design codes consider the shear key as a second defense line against unseating in seismic events with a task of preventing the deck from falling and limiting its transfer displacement to the provided clearance between the deck and the shear key. Furthermore, using additional shear keys in the top of the bent-cap is a common practice in Japan and the seismic exposed parts in Europe which are usually used for the same mentioned purpose.

Back walls should be considered a sacrificial part only when impact happens between the deck and the backwall; otherwise, both design codes recommend provisions to prevent impact and limit back wall damage, indicating that backwall is not sacrificial and not meant to be engaged during deck seismic response. To illustrate, EC8 [35] states that clearance between the deck and back wall should accommodate an appropriate fraction of the design seismic displacement and of the thermal movement so that damage under frequent earthquakes is avoided. For the shear keys, EC8 [35] states that seismic links (shear keys) and for all bearings types should be designed as capacity design effects (the horizontal resistance of the bearings shall be assumed to be equal to zero). JAR [59] design code requires for non seismically isolated bridges, in addition to the required seating length, using several provisions to control superstructure’s movement such as unseating prevention structure, which is in the bridge longitudinal direction; excessive displacement stopper in the transverse direction, which should be installed in all columns bents and abutments; and superstructure subsidence protection systems.

Some design codes, like JAR [59] seismic design code, recommend using seismically isolated bridges in which they direct the load path through the isolation bearings and the other devices
that are used to absorb energy and enhance bridge damping properties, which in turn concentrate damage in these parts. One of the most important requirements in this code is using devices that fully insure bridge unseating. Some seismic design codes in seismic prone areas in the US and Europe recommend using integrated bridges where all the bridge parts contribute in the seismic response. In these types of bridges, almost all the bridge parts are connected monolithically or semi-monolithically, and the bridge moves as one unit indicating that the load path passes through all parts and damage could develop in any bridge part. However, all design codes require that plastic hinges should happen in the piers and piles. As a result, design code assumptions regarding the load path and failure modes have a very important effect on both researchers’ and designers’ considerations in bridges’ design and analyses. This, in turn, will be reflected on the construction cost, analyses time and efficiency, and bridges’ assessment and clearly announce the significant effect of the load path and failure mode choice on bridges seismic demand prediction.

1.1.2 Design Criteria for Column Bents

Early bridge bents design theories relied upon the compression capacity of the beam-column joint. However, the importance of sufficient joint design had been overlooked leading to critical problems. Since the 1971 San Fernando earthquake and up to the 1994 Northridge earthquake, the then-adopted insufficient joint design resulted in catastrophic structural failure. Such failure led to different and complex design approaches and requirements, but without a unified methodology, that have been adopted across the U.S. and abroad. Some approaches results in weak designs, while other design approaches provided adequate strength but caused reinforcement congestion and construction problems. Studies on beam-column joint started since 1960s when Hanson and Conner [48] conducted several tests for the Portland Cement Association. Several joint load transfer mechanisms have been suggested based on Hanson and Conner [48] results. At that time, the most common mechanism depended on the compression resistance of the joint concrete. This
mechanize can be conceptualized by the formation of plastic hinges in the columns or beams, or both, that induce a significant shear in the joint causing diagonal shear cracks. Such cracking pattern develop diagonal compression struts that in turn will help in transferring the loads through the joint to the columns. This mechanism originated from beam column connection design for buildings, though, it was observed that the cap beam column connection in bridges behaves differently.

For beam column joint connection, in general, several design approaches were adopted by design codes. The differences between these approaches related mainly to requirements regarding the effect of confinement that is due to the loading and geometry, levels of reinforcement arrangement, and shear stresses and bond demands limitations. Examples of different design codes recommendations are the U.S. [20, 9, 104], New Zealand [100], and Japanese [7] recommendations. Further information about the differences between the mentioned design codes’ approaches can be found in Naito et al. [94]. The aforementioned mechanism, that depends on the resistance of the joint concrete, induced several problems such as limiting the columns ductility and contradicting the serviceability design criteria. In addition, it was found that the actual failure mode is sever shear cracking in the joint, joint deterioration, cap beam dilation and distress, and columns plastic hinge formation. Such failure mode did not satisfy the design codes requirements and induced several maintenance, serviceability, and construction problems. Accordingly, this approached was developed to the weak-column-strong-beam approach that provided high level of integrity to the cap beam-column region, that met the design limit state criteria and serviceability requirements. According to Caltrans [20], in the weak-column-strong-beam design approach, the columns should behave inelastically by developing plastic hinges at the ends while cap beams and superstructure should be a capacity protected components that should behave and maintain elastic behavior. Most design codes adopt the weak-column-strong-beam design approach, however, design recommendations and requirements for cab beam-column joint connection varies according to design codes’ objectives.
1.2 Bridges Modeling and Analyses

The main objective of bridges’ modeling and analyses is to determine bridges’ seismic responses represented by the members’ deformations, forces, and structural displacements. Several modeling approaches can be used in bridge modeling depending on the required responses such as the bridge structural systems and individual structural members. Bridge structural systems include the following:

(i) Global bridge models where most of the bridge components are represented in one global bridge model to determine the bridge’s seismic demands measurements using mostly linear elastic response spectrum analyses. These models have a limited usefulness and can be used only for short single span bridges, elastic response analyses, and basic ground motion analyses.

(ii) Frame models: These models are characterized by modeling bridge sections between the moving joints as a single or multi-frame model that are connected properly. A frame model offers an effective tool for the bridge seismic demands and capacity quantification, especially when pro-
viding detailed models, reasonable assumptions for the boundary conditions and the interactions between these frames and the adjacent frames.

(iii) Bent models: Bent models include modeling the columns’ bents or the abutments as individual subsystems and are usually used to determine the stiffness and rigidity that these subsystems provide to the bridge global models that will help providing more detailed and realistic models. Using the bent models also helps to include the boundary conditions’ effect, such as foundations or piles flexibility and the rigid body motion of the superstructure.

Individual structural members include all the forms of structural elements or members that can be used to model a global bridge model, frame model, or bent model in three or two directional form such as line or beam elements, plate or shell elements, and solid elements. These members can develop from simple elements with a single degree of freedom like a spring element to complicated elements with more than sixty degrees of freedom including displacement, pressure, and heat transfer degree of freedom.

Each of the aforementioned bridge structural models should be discretized to include their elements’ contribution and correlation with the other elements. Modeling discretization can develop from simple lumped models and components or subsystem models to the detailed finite elements models. Lumped models - where the structure is described as mass, stiffness, and damping that distributed in specific locations along the structural model - are simple mathematical models that require considerable knowledge and experience in modeling and representing the correct behavior. Subsystem models are characterized by developing an idealized subsystem model that contains several structural components to obtain each component’s effect on the overall behavior and the global subsystem response. Finally, in the detailed finite elements models, the structures are represented by a large number of small elements with high fidelity and complexity, trying to include all the components’ effect and capture all the possible structural responses and phenomena.
The analyses tools are the complementary part of the modeling procedure with an objective of evaluating the bridge seismic responses expressed by the bridge demand and capacity measurements. Analysis tools include static linear, nonlinear, P-\(\Delta\) analyses, linear response spectrum analyses, and dynamic nonlinear and P-\(\Delta\) analyses. Choosing the appropriate modeling and analyses tools for bridges depends on several parameters: the level of precision in the anticipated response, bridge design assumptions, bridge importance and precision in the required results, and degree of complication and detailing in the bridge modeling and whether it is going to be used in bridge seismic design or vulnerability assessment.

1.3 The Simplified Models

Simplified models are commonly used in bridges design and assessment processes. For new bridges, the simplified models are utilized in the design process to determine the seismic demands in terms of members’ sections’ strength and plastic hinge regions’ flexure strength. Several modeling simplifications are usually utilized in these simplified models that induce conservative design and exaggerate the structural demands. For instance, the simplified models use one dimensional (1D) elastic elements, that can not capture several responses, for the superstructure and bent cap for simplicity and neglect the abutment longitudinal resistance to maximize the structural displacement demands. The transverse abutment resistance is usually obtained using a simple approach or constrained for simplicity. Other assumptions are the neglect of several components contribution and phenomena such as abutments’ back wall and friction between the soil and structural parts, respectively. Finally, these models utilize linear elastic analyses procedures as recommended by design codes. Design codes [3, 20, 35, 58] recommend, for bridge design, using linear elastic static analyses (ESA) for displacement demands’ estimation of ordinary bridges and elastic dynamic analyses (linear elastic multi-model spectral analyses) for more sophisticated results where
the ESA method does not introduce sufficient level of sophistication to the estimate of the dynamic behavior.

For existing bridges, the simplified models are commonly employed in bridges’ assessment and determination of the deformation capacity of individual bridge components, groups of components, or a complete bridge system. Such models should be sufficient to capture most of the bridge’s local and global responses. It should be composed of every structural element or member with an accurate representation of its geometry, behavior, material constitutional relationship, nonlinear response, contribution to the resistance, and its correlation with the other components.

Such complicated task that include developing efficient models and predicting reliable bridges’ behavior need an experience in each bridge component and its contribution to the global behavior. In addition, to obtain the most accurate results, design codes recommend using nonlinear analyses procedures such as static pushover and time history analyses for structural displacement capacity and bridge components demands prediction. Yet, studies continue to use further simplifications to reduce the complexity of the problem even though the results are still least plausible and imponderable when using these simplifications.

For instance, abutments behavior, embankment flexibility, and soil structural interaction have a critical impact on the entire bridges’ seismic response and functionality during and after earthquakes. These effects that are usually implicitly considered in the load path and failure modes assumptions, have been modeled, analyzed, and studied by several studies with different complexities starting from roller or hinge boundary conditions and springs and dampers systems to several components systems or detailed finite element models. Capturing the correct load path, though, requires efficient sophisticated modeling that includes all components’ contribution and their resistance mechanisms, tridirectional effects, soil structure interaction, and the components’ local behavior that includes their mass, stiffness, and nonlinear behavior. Though, simplified models
are still being used to study such complicated responses, through using simple mathematical representation, design codes hypotheses, and neglecting several components’ contribution, losing the actual load path and failure mode and leading to unpredicted damage, overestimated bridge seismic demands, or incorrect understanding of bridge behavior.

1.4 Research Objective

The constantly used simplified modeling and analyses procedures that are commonly adopted in bridges’ design and assessment procedures do not usually capture the correct load path and failure modes. In particular, when using the one-dimensional elements in a simple 3-D model, such as the spine model. This defect is due to the incorrect load path and failure modes assumptions used in such models. As a result, these models will have a drawback effect on bridges’ seismic demand and capacity prediction, which in turn affect the analyses and design efficiency and understanding of bridges’ behavior.

For these reasons, further identification to the impact of such assumptions and simplifications is required.

The main objective of this research is to quantify the correct load path and failure modes that should be used in bridges’ simplified models and their consequences on the modeling and analysis outcome. The effect of the simplified models on bridges’ seismic demand prediction is also presented in this study. The first part of the objective was conducted through the use of detailed three-dimensional finite element models (FEM) that were compared with a paralleled simplified models to identify the deficiency resources. Three levels of modeling and analyses were performed. In the first level, several bridge components were modeled and analyzed to identify the effect of using simplified models on the behavior of the components and verify the adequacy of the used
materials models and analyses procedures to predict a realistic component’s response. The second level utilized two bridge subsystems that were modeled and analyzed using static pushover analyses to determine the contribution of each component to the subsystem resistance, identify the effect of the simplified models on the subsystems’ behavior, and determine the simplified shortcoming in predicting the actual response. In the third level, two approaches were employed in which a set of single degree of freedom systems and 3-D full bridge systems was utilized. In these approaches, nonlinear time history analysis was used to determine the effect of four abutment parameters on the system response. These parameters are the abutment stiffness and strength in the longitudinal and transverse direction. In the full bridge approach, a Probabilistic Seismic Demand Analysis was used to determine the effect of the abutment parameters. The main focus of this research is to characterize the shortcoming in the currently used load paths and failure modes for better bridges’ seismic demand estimate, not their design capacity evaluation.

1.5 Research Plan

This research includes five chapters. The first chapter, as seen, introduces the most common modeling and analyses techniques, simplification, and assumptions that are recommended by most design codes and followed in current design practice. This chapter also introduces the design codes’ assumptions regarding the load path and failure modes in bridges and the hypotheses that should be used in bridges’ components design.

The second chapter includes a review of the modeling and analyses procedures that were employed in research and the disadvantages of these procedures, according to others’ conclusions, comparing to more complex and sophisticated procedures such as the continuum finite elements analyses. A review of the most common bridge failure modes and damage that have been observed as a result of real events in addition to their failure mechanism and predicted reasons was also introduced in
the second chapter.

The next three chapters introduce several levels of modeling and analysis complicity and sophistication which start from the component level where a static analysis of a single component was considered to a full nonlinear dynamic analyses of a full bridge system. In chapter three, several critical bridge components were modeled and analyzed using both packages. The bridge components that were analyzed in this chapter are a bridge pier, shear key, abutment backwall and embankment, T-girder beam, and laminated elastomeric bearing pad. These models were used to identify the shortcoming in the simplified models to predict the correct response in the component level. The results from the components analyses were then compared with the experimental results for each case to determine the ability of the simplified modes to predict a realistic response and verify and calibrate the finite elements models to provide better agreement with the experimental results.

In the fourth chapter, the verified materials’ models and modeling and analyses techniques were utilized to study two bridge subsystems. These subsystems include abutment and column bent subsystems. Each subsystem was modeled and analyzed using both packages. Besides, several cases for the subsystems were considered such as different abutment configuration, boundary conditions, and cap beam torsional and flexure stiffness. Static pushover analyses procedure in the longitudinal and transverse direction were used to investigate the actual load path and failure (damage) modes in each bridge subsystem.

Finally, in chapter five, two approaches were used to determine abutments’ properties effect in the full bridge level. In the first approach, a set of single degree of freedom (SDOF) systems were first developed to represent the response in each direction. These SDOF systems were then utilized in a nonlinear time history analyses to investigate the abutment subsystem findings and determine the effect of abutments’ parameters on bridge’s response. Next, in the second approach
and to include the multi-directional effects and to provide a better estimation of the actual system response, a full bridge system was modeled and utilized in a nonlinear time history analyses for the same objective. A Probabilistic Seismic Demand Model (PSDM) that uses Probabilistic Seismic Demand Analyses (PSDA) approach was applied and used in the 3-D analyses to confirm the SDOF systems’ results. The abutment’s parameters that were studied are the abutment stiffness and strength in the longitudinal and transverse directions. The results revealed the effect of modeling simplifications and impact of abutments’ parameters on bridges’ seismic demand and capacity prediction.
CHAPTER 2: BACKGROUND STUDY

2.1 Introduction

Bridges are one of the most critical structural systems that have sensitivity to several problem areas. Some of these problem areas are structural soil interaction with bridge systems, bridge earthquake response, load path (force transfer mechanism), bridge failure modes, and damage sequence of the bridge components. Studying such problem areas may be achieved by considering a full bridge systems, bridge subsystems that consist of several bridge components, or studying each bridge component separately and approximate the total behavior. Bridge’s components behavior is a popular area in which extensive research has been done. Full bridge systems and bridge subsystems have been studied by many researchers, however, several assumptions and simplifications are commonly used due to the problems complexity and excessive computational time required to study such problems which leads to considerable deficiencies in such procedures. Typical bridge components are classified into three main categories. The first category is the superstructure components that consists of girders and/or diaphragms, slab deck, and barriers. The second category is the substructure components that consist of abutments, bents, and foundations. Abutment consists of stem wall, backwall, wing walls, backfill and embankment, and shear keys. Bents consist of cap beam and piers while the foundations consist of footing or pile cap and piles. The third category of the components is the bearings that connect the superstructure and substructure components. A typical bridge components and its parts is shown in figure (2.1).

Understanding and predicting bridge behavior in real events can be achieved by one of the following ways; (i) experimental tests on a full bridge, scaled bridge, bridge components assembly, or single bridge components, (ii) developing bridges’ analytical models and validating the results with experimental tests that already have been carried out, (iii) developing analytical models of
the system under study and comparing with other analytical studies. Bridge analytical models can be categorized into two types depending upon the complexity and degrees of freedom considered in the analysis that are: two-dimensional (2D) and three-dimensional (3D) analyses models. The selection of the analyses type depends on the bridge response under consideration and whether the longitudinal or transverse direction was considered as the controlled direction. Some studies indicated that the longitudinal direction controls the response of the bridge [25, 111, 132] while others indicated that the transverse direction controls the bridge damage [8, 24]. On the other hand, some studies believe that the bridges should be modeled in three-dimensional models with some complexity, combining the longitudinal and transverse direction in order to represent the system sufficiently [61, 144].
2.2 Full Bridge Analytical Models

Since the experimental test of a full or components bridge system is highly expensive and needs experience in instrumentation and testing in addition to the availability of the test bridge, investigating bridge responses using analytical models is the most common and promising technique. Early researchers used simplified methods to study bridges’ behavior. Finite difference, grillage method, beam stick, and simplified finite element method are some of these basic analyses techniques. Chen et al. [23] used finite difference method to analyze skew bridges taking into account girder to slab stiffness ratio, girder spacing, and span length. Calculating the moment and deflection coefficients were the objective of this study. Hendry and Jaeger [50] utilized their grid-frame method to estimate load distribution factors for skew bridges. Mehrain [89] developed a finite elements computer program to analyze skewed composite bridges and employed different elements type to study the convergence.

Spine or beam stick model is one of the most common methods that had been used by early researchers. Gustafson and Wright [46] introduced a new finite element method that use eccentric beam elements and parallelogram plates to analyze two typical composite skew bridges and investigate skewness and midspan diaphragms effect. Ghobarah and Tso [43] used a special beam model that is capable of deformation in the flexural and torsional direction to model Foothill Boulevard Undercrossing, S.E. Bridge under earthquake excitation. The study showed that the coupled flexural and torsional vibrations may be the major damage mechanism for all California skewed bridges. DeCastro et al. [30] employed 120 finite element models of I-beam superstructure with different length, width, and number of beams to propose correction factors for normal bridges’ load distribution factors in order to estimate the skewed bridges’ distribution factors.

Using a commercial software such as ABAQUS [5] is one of the techniques that can be used to represent a full structure or its components in two or three-dimensional model. These packages can be
used to model the bridges with a high degree of fidelity by considering as much components’ contributions as desired. However, it requires an extensive running time and sometimes these models cannot be run due to its complexity even with high specification computers. Still, the continuum finite elements models is one of the best competitors that can be used to capture several phenomena and the tridirectional effects. Abdel-Mohti and Pekcan [6] concluded that a detailed finite element model is required to capture the higher modes effect for bridges with skew angle larger than 30 degrees. Nouri and Ahmadi [99] emphasize that a 3D finite element model is essential for bridges with skew angle over 20 degrees due to the considerable change in the bridge responses. Another common three-dimensional modeling makes use of simplifications such as spine or stick model of structures, which widely used in finite elements frameworks such as OpenSees [102].

The 2D and 3D models that can be used to represent bridges’ structural characteristics, depending on the required accuracy and time efficiency, could be a single degree of freedom (SDOF) models, multi-degree of freedom (MDOF) models, and continuum finite element models (FE). The SDOF model advantages are the low computational demand and expectable global bridge behavior, but it loses the local behavior and three-dimensional effect. Beam stick models or MDOF models, on the other hand, can capture the tri-directional effect but it still loses the local behavior. The most promising models that capture the local and global structural behavior of bridges under dynamic loads is the detailed finite-element model which is the most preferred model despite its complexity and computational time.

As mentioned before, full bridge and bridge subsystem models provide better realistic response and can capture several phenomena. Though, the assumptions and simplifications that are regularly used due to the problems complexity and excessive computational time induce considerable deficiencies in such procedures. Such deficiencies, that have been observed in real events, include inaccurate prediction of the bridge performance, unanticipated local and global bridge failure, and unexpected components damage. Several reconnaissance reports indicated unexpected local and
global bridge damage, due to the modeling and analyses simplifications and design codes hypothe-
ses adopted in the design, such as superstructure unseating [19, 38, 67], shear key and backwall
failure [67, 122, 63, 145], excessive abutment foundation displacement [60, 122], excessive defor-
mation of embankment and backfill soil [67], superstructure displacement and rotation [79, 44],
and column bent failure [90].

Analytically, due to the deficiencies that were observed due to the use of modeling simplifications
and assumptions, several studies emphasized the necessity of detailed finite elements models, im-
portance of load path assumptions, and consequences of modeling assumptions and simplifications.
McCallen and Romstad [84] used beam stick element to represent the bridge’s box-girder, piers,
bent cap, and backwall in a short span overpass bridge model. The authors needed to add artificial
stiffness to the cap beam by increasing the bending properties due the excessive flexibility and
deformation that observed in the behavior due to the use of the beam stick model. They concluded
that the stick model induce excessive flexibility and deformation in the cap beam and superstructure
and the finite element model was more promising in capturing the mode shapes and deformations.

Saadeghvaziri and Yazdani-Motlagh [119] studied a multi-span simply supported bridge using 2D
and 3D spine models. The effect of modeling approach and appropriateness of pushover analy-
sis was investigated considering the stiffening-interaction between the bridge and the abutments.
Several nonlinear time history analyses were performed resulting to several conclusions. Some
of these conclusions are the sensitivity of the bridge to soil-structure interaction and necessity of
the three-dimensional models to represent the coupling in the orthogonal direction and capture the
effect of the piers’ axial load variation. Aviram et al. [10] created six spine models of a typical Cal-
ifornia reinforced concrete box girder bridges to develop modeling recommendation for bridges’
nonlinear analysis under seismic loading. Two software packages were used that are SAP2000
and OpenSees. In the two software packages, similar modeling assumptions were used for the
superstructure, cap beam, and abutments while different modeling assumptions were used for the
column bents. The authors found discrepancies, especially with high intensities, between the two models which were attributed to the differences in the elements formulations for the utilized software packages.

Several studies confirm the efficiency of 3D continuum finite element models versus simplified models. Huo [55] utilized a 3D finite element model to calibrate $p - y$ springs for several abutment wall heights and embankment soil properties. Both, the longitudinal and transverse abutment effect were included in addition to the gap and back wall-back fill friction. The conclusions included the importance of the abutment modeling approach and its significant effect on the longitudinal and transverse bridge response, considerable effect of the analyses assumptions and simplifications, and the controlling role of the abutment type and configuration on the bridge seismic resistance.

A 3D continuum finite element model was also created by Rahmani et al. [107] for the Meloland Road overpass. Eight node brick elements and four nodes shell element were used to model the foundation system while elastoplastic elements were used for the piles. The other bridge parts were modeled as elastic elements following the capacity protected design assumption. After comparing the FEM and actual bridge responses, the authors concluded for complete bridge systems, the continuum models is an effective tools for detailed analysis, but requires high performance computing and timely analysis. Zhang et al. [149] and Elgamal et al. [37] used 2D and 3D models to study soil-structure interaction on bridges’ behavior and emphasized that a 3D model should provide more realistic and accurate results.

### 2.3 Bridges’ Subsystem Models

One of the techniques that can be used to study bridges’ behavior is subdividing the bridge into subsystems such that assembling their response can represent the total bridge response. Subsystems consist of bridge parts that share the stress path (forces path) and transfer the forces between
the major bridge portions. An abutment subsystem, as an example, would consist of shear keys, stem wall, wing walls, bearings, back wall, piles and pile cap, and back fill and embankment soil. All these parts contribute in transferring the forces from the bridge superstructure to the ground. Modeling such system can be made in a very simple approach such as three-dimensional springs model or a complicated approach like 3D continuum finite element model for each part of the system and combining them in a single complete model. Examples of bridges’ subsystems are abutment subsystem, column bent subsystem, and superstructure subsystem. Since this research is interested in bridges’ abutments and column bents, only studies on abutment and column bent subsystems will be reviewed.

2.3.1 Abutments’ Studies

Studies on abutments’ can be trace back to the seventeen century. Coulomb [27], Rankine [110], and Terzaghi [141] methods were the earliest methods for estimating the earth pressure against walls. Failure surfaces considered were wedge and log-spiral surfaces. Janbu [57] and Shields and Tolunay [131] extended Terzaghi [141] method to account for vertical and horizontal inter-slice forces. Extensions to passive earth pressure to account for dynamic interaction of wall and soil were made by Martin et al. [82] and Shamsabadi et al. [129]. Experimentally, extensive work has been conducted by many researchers in this area such as Rowe and Peaker [118]; James and Bransby [56]; Fang et al. [39]. Large scale tests were also conducted in this area by Maroney [81]; Rollins and Sparks [115]; Cole and Rollins [26]. The original assumption for the previous studies is the wall solo existence that is not connected to other components. As a result, this assumption originated the current design practice hypotheses that consider the backwall as sacrificial, back fill as the only resistance to the superstructure movement, and neglect the other components’ contribution.
Abutment properties (stiffness, capacity, and force-deformation hysteresis) are essential parameters commonly utilized to represent the behavior of abutment-soil systems. Researchers adopted several approaches to provide sufficient approximation for these parameters. Several analytical models were developed to estimate the stiffness using abutment dimensions and/or soil properties [143, 71, 135]. Other studies used recorded data from instrumented bridges to estimate the stiffness [44, 84, 142, 80, 116]. Zhang and Makris [148] developed an equivalent nonlinear approach for deriving the embankment impedance, stiffness, and damping. Shamsabadi et al. [127] proposed a method to predict the force-displacement capacity of a bridge abutment-embankment system for seismic design. Sextos et al. [125], and later Taskari and Sextos [139] studied several typical California abutment-embankment systems to derive force-deflection relationships. Experimental studies were also used to estimate abutments’ properties such as Crouse et al. [29], Douglas et al. [32], Romstad [117].

Based on performance-based seismic design, an accurate representation and prediction of the abutment behavior at the system and component level is essential. Several modeling approaches, simplifications, and assumptions were used by studies to understand abutments’ behavior. Shamsabadi et al. [130], Nielson [96], and Nielson and DesRoches [97] proposed several models to investigate abutments’ effect on bridges. Their models included different approaches such as gap elements, dashpots, and linear and nonlinear springs. However, most of them neglected the back wall and shear key resistance in the longitudinal and transverse direction, respectively. Many studies emphasized on the significant effect of abutment modeling approach, boundary condition configuration, and abutment assumed properties on bridges behavior. Aviram et al. [11] found that using simplified abutment models cause major differences in the ultimate shear strength, displacements, mode shapes, and periods and concluded that it is consequential to choose appropriate abutment model to capture the correct response. Zhang et al. [149] and Elgamal et al. [37] used 2D and 3D models to study soil-structure interaction on the bridge behavior and emphasized that a 3D model should
provide more realistic and accurate results.

A comprehensive treatment for soil-structure interaction and improved the modeling approaches for bridges components were proposed by Omrani et al. [101]. The authors concluded that the existing design codes provisions significantly underestimate the shear keys capacity and the backfill response has a significant impact on bridge response. Luo et al. [73] confirmed the high importance of modeling sophistication on the force-transfer mechanisms, damage sequence, components’ fusing performance, and failure modes. By comparing a simplified and detailed abutment models, it was found that the simplified models cause significant overestimation of the piers and superstructure displacement, does not capture the foundation displacement, and largely underestimate the abutment forces which results in a misunderstanding of the bridge seismic response. They also concluded that the back wall does not break even under maximum earthquake hazard level which can alter the load path between the abutment and the superstructure, cause excessive abutment displacement, and induce early occurrence of several other limit states. Recently, Mackie et al. [74] perform sensitivity analyses to investigate the boundary conditions and abutment parameters effect on bridges’ behavior. They concluded that the boundary condition choice of the springs and/or single point constrains at the deck derive the elastic and inelastic analysis responses.

Many studies emphasized the significant effect of abutments’ shear keys on abutment behavior in the transverse direction. Shear key behavior, failure mechanisms, and effect of their design hypotheses on the behavior were investigated by Priestley et al. [106], Bozorgzadeh et al. [17], and Bozorgzadeh et al. [18]. Their results showed that the adopted design hypotheses affected the shear key failure modes. Goel and Chopra [45] investigated the shear key role on the seismic behavior of regular bridges. It has been found that due to the shear key conservative design (maintained elastic behavior), the seismic demand of a bridge with nonlinear shear key can be limited between the demands of a bridge with elastic shear keys and a bridge with no shear keys. The effect of shear keys modeling approach was also confirmed by Kaviani et al. [62] that showed major changes in
bridges’ demands due to shear keys failure status. Real events also confirmed the effect of shear key design hypotheses on bridges’ behavior. Several observations indicated the propagation of the shear key failure to the stem wall, unpredicted system response, and undesirable behavior that is due to the shear key design approach (Priestley et al. [106], and Yashinsky et al. [145]).

2.3.2 Column Bents’ Studies

Many studies were investigated columns bents. First studies concentrated on the problems in the cap-beam-column joint region. Such problems include joint confinement, bond slip, load path, failure mode and mechanism, and effect of the superstructure on the cap beam stiffness and strength. The failure mode that was adopted by design code is the weak-column-strong-beam approach that assumes the inelastic behavior of the columns and superstructure and cap beam elastic behavior. It was observed in the experimental tests that the actual is the cap beam-column joint cracking, cap beam dilation, and cap beam-column joint distress and deterioration. Several problems have resulted due to such failure mode, despite the plastic hinge formation in columns ends, that include difficulties in post-earthquake repairs and limiting the columns ductility due to cap beam and joint deterioration. Accordingly, further studies were conducted to investigate the actual failure mode, enhance the design codes revisions to ensure satisfying design codes objectives. The resulted recommendations included increasing the cap beam width and strengthening the cap beam-column joint by increasing its recommended reinforcement.

In the first cap beam-columns joint studies, only uni-directional loading tests were conducted that is attributed to the difficulty of these tests and the nonavailability of the equipment required for such tests. Seible et al. [123] and Seible et al. [124] are among the first studies on cap-beam columns joints. Full-scale cap beam-column joint test was performed to investigate the actual failure mechanism of the joint and effect of the obtained failure mechanism on the global seismic
response. It was found that the failure mode is significant cap beam-column joint shear cracking and cap beam dilation in the longitudinal direction. According to the test results, several design recommendations were suggested by the author that insure damage repair without traffic interruption. The failure mode, strength degradation, and peak strength were also investigated by MacRae et al. [76]. A three-quarters large-scale test of the Santa Monica Viaduct PC cap beam-column joint was tested under reversed cyclic loading. The author also concluded that the main sub-assemblage failure is due to joint shear failure and recommended design approaches.

Mazzoni and Moehle [83] conducted one of the first bi-directional tests on beam-column joints in bridges. The authors investigate the seismic response and design of the double deck reinforced concrete bridges. A better understanding of the load path and residual stresses in the bridges mentioned above’ type under bi-directional loading. Naito et al. [94] investigated the difference between different design approaches by testing four large-scale bridge beam-column joints. They concluded that design codes recommendation adopted at that time induce elastic response for the cap beam-column joints and inelastic (plastic hinges) in the columns ends.

Mosalam et al. [92] extended Naito et al. [94] tests by considering full-scale bridge sub-assemblies that consist of three columns bridge bent that is monolithically connected to a large segment of cast-in-place box girder superstructure. The test specimens were also modeled using detailed finite elements models with special attention to the boundary conditions and effect of the bi-directional loading on the global response of the global superstructure damage. The observed damages were cap beam-column joint shear damage and reinforcement pullout for the longitudinal reinforcement. The author concluded that using headed bars would in the cap beam column joint provide better local and global performance. Moustafa and Mosalam [93] tested two large-scale cap beam-column bent sub-assembles that included column, cap beam, and superstructure segment. Also, one-dimensional, two dimensional, and three dimensional FEM were created for the full bridge and test specimen to investigate the column-bent cap beam-box girder sub-assembly behavior and
estimate the contribution of the deck slab and soffit under combined vertical and lateral loading. One of the extracted conclusions is that the observed failure mode satisfies all Caltrans SDC objectives.

2.4 Damage of Bridges in Earthquakes

The observed bridges behavior and response during real earthquake events provide an important opportunity to understand and predict a realistic bridge seismic response, failure mode, and damage accumulation that can be adopted in bridges’ design and analyses. The most common bridges failure modes and damage that have been observed as a result of real events in addition to their predicted reasons are:

2.4.1 Failure Mechanism in regular Bridges

1. Column Failure: Several deficiencies can cause column failure. It could be related to the bridge geometry, such as skew angle and columns length, boundary conditions, or the design philosophy and the reinforcement details that yield from such philosophy. Columns failure that is related to design philosophy includes ductility and flexural strength failure and column shear failure. Flexural strength failure includes insufficient flexural strength that results from low seismic design lateral forces, unreliable plastic hinges’ flexural strength that can be a consequence of short splice length, inadequate flexural ductility which can be defined as the ability to deform inelastically for several cycles without excessive strength degradation, and early columns reinforcement cut off that causes flexure-shear failure. The second column failure is the shear failure which usually takes place when the flexural strength surpasses the shear strength. These failure stages develop from excessive shear cracking followed by transverse reinforcement yielding or fracture which cause buckling or bowing of longitudinal
reinforcement and ends with a complete loss of columns shear strength. Column shear failure may be considered one of the most important failure types in bridges seismic response due its recurrence over the years. Examples of columns shear failure can be found in the 1971 San Fernando, 1987 Whittier, 1994 Northridge, 1995 Kobe earthquakes and more.

2. Seismic Displacement Amplification: This type of failure characterized by the inadequate seismic displacements that are adopted in bridges seismic design. This type of failure usually happens in bridges that are designed before 1970 where bridges seismic design is based on gross section stiffness, elastic design, and low lateral load level. These deficiencies induce a considerably short adjacent structures spacing and seating length. This insufficient seating length or spacing can cause three types of failure which are: (i) Span Unseating at the movement joints which is due to spans movements relative to each others in the longitudinal direction. (ii) Displacement magnification due to soil conditions: this type of failure is due to special soil conditions such as soft soil and liquefiable soil that magnify the bridge structural vibrational response that in turn causes unseating. (iii) Bridge structures pounding which is a consequence of unsatisfactory spacing between bridge adjacent structures and usually occurs in adjacent structures with different heights or skewed bridges. The aforementioned failure modes represent the best example for the effect of the modeling simplifications and the inaccurate damage and failure modes that are adopted in the design proses such as low level of seismic load (incorrect damage and failure modes) and elastic components behavior (modeling simplification and improper design hypotheses). The consequences of these assumptions would be underestimation of the system strength and unpredicted behavior that would be more pronounced in seismic events.

3. Beam Cap Failure: this failure can result from three types of deficiencies that are: inadequate shear capacity, early negative moment reinforcement cut-off, and insufficient anchorage reinforcement between the cab beam and the other parts. All the mentioned deficiencies could
be a result of incorrect load path or failure mode assumptions that are considered in the design hypotheses. The improper modeling that is a consequence of modeling simplification is another possible reason for these deficiencies. For example, besides the underestimation of the actual strength, the incorrect assumption of the load path would result in unpredicted failure mode and undesirable behavior that is due to damage concentration in the critical components.

4. Cap beam-columns joint failure: this type of failure is a common failure for most bridges that have improper seismic shear design or low importance seismic design that are commonly considered for these joints, especially for old bridges. In fact, this failure is a common result of the incorrect damage and failure mode that is adopted by the design codes. As motioned above, old bridges that were built before 1970, suffer from insufficient seismic design that results from modeling simplifications and improper failure mode assumptions. The outcome of these assumptions would be shifting the failure mode causing unpredicted bridges’ response.

5. Prestress Concrete Girders Failure: This type can result from inadequate flexural or shear strength of the prestressed girders. One of the common failure modes in prestressed girders bridges, especially for spans without diaphragms, is spans’ collapse due to girders insufficient strength or girders unseating due to side stopper failure. Romero Bridge failure, which due the 2010 Chile earthquake, is a good example for this type of failure [63]. This failure type is a good example for the design code hypotheses regarding the load path and sacrificial shear key assumption that should provide ductility to protect the girders from unseating. Girders failure could also result due to the shear keys excessive strength that is not considered in the design proses that would alter the load path concentrating the damage in the prestressed girders.
CHAPTER 3: BRIDGE COMPONENTS ANALYSES

3.1 Introduction

Studying bridges structures using a full bridge system or several subsystems requires using various modeling and analyses tools. These tools include several elements types, materials constitutive models, solvers and solutions controls, analyses procedures, convergence criteria and many other techniques that help to achieve the correct response. The first step in the modeling and analyses process is a verification procedure that should be performed to prove the ability these tools to provide realistic behavior. Utilizing the correct material constitutive models, parameters, and its capability to sufficiently represent an accurate material behavior is the most important requirement in the verification process. This chapter will include the concrete plasticity models that will be used in the analyses, the analyses procedures, the concrete and steel constitutive models, and several bridge components modeling and analyses that will be utilized to identify the shortcomings of the simplified models to predict the actual component response and verify the ability of the adopted FEM to provide the correct components’ actual behavior and failure mode. The components will be modeled in the finite elements software ABAQUS [5] and open source package OpenSees [102] to verify the adequacy of the used materials models and analyses procedures to predict a reliable components response. The finite elements analyses results will be compared with a corresponding experimental tests of the components under study to differentiate the deficiencies of the one-dimensional model versus the three-dimensional continuum finite element model in capturing a realistic component local behavior.
3.2 Abaqus Material Models

There are three concrete models to model the behavior of the concrete material in Abaqus. Smeared cracking model, concrete damage plasticity model CDP, and brittle cracking concrete model (ABAQUS [5]). Due to the use of the concrete damage plasticity model CDP throughout this research. A summary of the CDP model will be presented:

3.2.1 The Concrete Damage Plasticity Model

The concrete damage plasticity CDP model can be used to model concrete structures and other quasi-brittle material structures. The CDP model use isotropic damage plasticity that considers isotropic tensile and compression plasticity. The model was designed to work for several concrete loading conditions such as monotonic, cyclic, and dynamic loading conditions. CDP mode is a continuum plasticity damage model that consider the concrete crushing in compression and cracking in tension as the main failure criterion. Figure (3.1) shows the concrete behavior in tension and compression.
3.3 Concrete Plasticity

3.3.1 Yield Surface

Lubliner et al. [72] yield function that was modified by Lee and Fenves [68] to take the strength evolution under tension and compression under consideration is adopted in the concrete damage

Figure 3.1: CDP model concrete behavior in tension and compression (ABAQUS [5])
plasticity model. The yield function has the form

\[ F = \frac{1}{1 - \alpha} (\bar{q} - 3\alpha \bar{p} + \beta (\bar{\epsilon}^{pl}) \langle \hat{\sigma}_{max} \rangle - \gamma (\langle -\sigma_{max} \rangle \rangle - \bar{\sigma}_c (\bar{\epsilon}^{pl}) = 0 \]  

(3.1)

where

\[ \bar{p} = \frac{1}{3} \bar{\sigma} : I \text{ is the effective hydrostatic pressure,} \]
\[ q = \frac{2}{3} \bar{S} : \bar{S} \text{ is the Mises equivalent stress,} \]
\[ \bar{\sigma} = \bar{\sigma} I + \sigma \text{ is the deviatoric part of the effective stress tensor} \]

\[ \alpha = \frac{(\sigma_0/\sigma_c) - 1}{2(\sigma_0/\sigma_c) - 1}; 0 \leq \alpha \leq 0.5, \]  

(3.2)

\[ \beta = \frac{\bar{\sigma}_c (\bar{\epsilon}^{pl})}{\bar{\sigma}_t (\bar{\epsilon}^{pl})} (1 - \alpha) - (1 + \alpha), \]  

(3.3)

\[ \gamma = \frac{3(1 - K_c)}{2K_c - 1} \]  

(3.4)

The hardening variables \( \bar{\epsilon}^{pl} \) and \( \bar{\epsilon}_t^{pl} \) which have the main effect on the yield surface evolution was expressed in the CDP as an equivalent effective plastic tensile stress (PEEQT) and equivalent effective plastic compression stress (PEEQ), and

\( \hat{\sigma}_{max} \) is the maximum principle effective stress; \( \sigma_0/\sigma_c \) is the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress with a range of (1.10 - 1.16) (Lubliner et al. [72]); \( K_c \) is the ratio of the second stress invariant on the tensile meridian with a range of 0.5 \( \leq K_c \leq 1.0; \) \( \bar{\sigma}_c (\bar{\epsilon}^{pl}) \) is the effective tensile cohesion stress; and \( \bar{\sigma}_t (\bar{\epsilon}_t^{pl}) \) is the effective
compressive cohesion stress. Figure (3.2) shows the yield surface of the CDP model.

![Diagram of yield surface](image)

**Figure 3.2: Yield surfaces in the deviatoric plane and plain stress (ABAQUS [5])

3.3.2 Flow Rule (Plastic Flow)

Non-associated potential plastic flow is used in the CDP model. The flow potential $G$ in this model is the Drucker-Prager hyperbolic function:

$$G = \sqrt{(\epsilon \sigma_{t0} \tan \psi)^2 + \bar{q}^2} - \bar{p} \tan \psi$$  \hspace{1cm} (3.5)

where

- $\psi$: is the dilation angle in degrees measured in the p-q plane at high pressure.
- $\sigma_{t0}$: is the uniaxial tensile stress at failure.
- $\epsilon$: is the eccentricity that define the rate at which the flow potential become a straight line and the
eccentricity approaches zero.

The default value of the eccentricity is 0.1 which means that the material has the same dilation angle over a wide range of the confining stress. A smooth and continuous flow potential was considered in the CDP model to ensure the flow direction uniqueness.

![Hyperbolic Drucker-Prager flow potential](image)

**Figure 3.3: The Drucker - Prager hyperbolic plastic potential function (ABAQUS [5])**

In concrete damage plasticity model, the recovery of the concrete compression and tension stresses during loading cycles are considered by using the weight factors $\omega_c$ and $\omega_t$ which corresponds to compression and tension recovery, respectively. These weight factors were assumed to be material properties. It should be equal to one in the case of full recovery and zero for none recovery. The typical uniaxial load cycle for tension and compression that considered in the CDP can be seen in figure (3.4).
3.3.3 Hardening Rule

Linear Drucker-Prager yield surface and flow in the deviatoric plane is shown in figure (3.5). The associated flow rule of CDP model considers the material friction angle equal to the material dilation angle. In CDP mode, for granular material (e.g., concrete and rock), nonassociated flow rule were adopted which assumes that flow is normal to the yield surface in the II plane and the dilation angle $\psi$ is smaller than the material friction angle $\beta$. 

Figure 3.4: CDP typical uniaxial cycle load for tension and compression (ABAQUS [5])
3.4 Concrete Material Model

The required data sets for the concrete damage plasticity are:

(i) The uniaxial compression stress strain curve and the uniaxial tension stiffening curve which can be represented as the stress-strain, stress-crack width ($\omega$), or stress-fracture energy GF curve.

(ii) The compression stress-damage curve and the tension stress-damage curve.

(iii) The concrete damage plasticity parameters which are: the dilation angle $\psi$, the eccentricity $\epsilon$, the ratio of the initial biaxial compressive stress to uniaxial compressive stress $\sigma_{0b}/\sigma_{0c}$, the ratio between the compression to tension meridian of the second stress invariant $K_c$, and the viscosity parameter $\mu$.

All the concrete material in this research will be modeled using the same constitutive material model. The concrete uniaxial compressive stress-strain curve was created using the stress-strain relation proposed by Carreira and Chu [22]. The concrete crushing failure in compression will
be modeled using the concrete compression damage criteria which represented by the relation between the inelastic strain and compression damage factor. The concrete compression damage factor will be calculated according to Birtel and Mark [15] where the plastic strain was assumed to be equal to 70% of the inelastic strain and the nonlinear part of the compression stress-strain curve starts at a stress equal to 40% of the maximum compressive strength. Another assumption is the residual concrete compressive strength which assumed to be 20% of the concrete compressive strength $f'c$ in order to avoid any computational or numerical problems in the analyses. The maximum tensile strength for the concrete materials in the study were taken equal to $(6\sqrt{f'c})$ in MPa. The concrete tension damage factor was calculated according to Birtel and Mark [15] where the plastic strains were assumed to be d 10 % of the inelastic strain in tension. The mentioned assumptions were based on Reinhardt and Cornelissen [112] for tension and Sinha et al. [136] for compression. Carreira and Chu [22] proposed compression stress-strain relationships that have been used in this study are shown below:

$$\frac{f_c}{f'c} = \frac{\beta(\epsilon/\epsilon')}{(\beta - 1 + (\epsilon/\epsilon')^\beta)}$$ (3.6)

$$\beta = \frac{1}{1 - \frac{f'c}{\epsilon'_c \epsilon_{it}}}$$ (3.7)

when only $f'c$ is available $\beta$ can be found from using

$$\beta = \left(\frac{f'c}{4.7}\right)^3 + 1.55$$ (3.8)
or the ratio \( \frac{f'c}{E_{it}\epsilon'} \) can be obtained according to Saenz [120]

\[
\frac{E_{it}\epsilon'}{f'c} = \frac{31.5 - (f'c)^{1/4}}{(f'c)^{1/4}[1 + 0.006f'c]^{1/2}}
\]

(3.9)

or according to Hognestad [52]

\[
\frac{E_{it}\epsilon'}{f'c} = \frac{3600}{f'c} + 0.92
\]

(3.10)

when the chord modulus of elasticity \( E_c \) is available \( \beta \) can be determined using

\[
\alpha \left( \frac{f'c}{E_c\epsilon'} \right)^\beta - \beta \left( \frac{f'c}{E_c\epsilon'} - 1 \right) - 1 = 0
\]

(3.11)

where \( \alpha = 0.4 \) when \( E_c \) is determined according ASTM C 469

when the concrete unite weight \( \omega \) is available, the ratio

\[
\frac{f'c}{E_c\epsilon'} = \left( \frac{(f'c)^{0.7}}{158.1\omega^{1.51}\epsilon'} - 1 \right)
\]

(3.12)

equation (3.12) would be substituted in equation (3.11) to find \( f'c/E_c\epsilon' \), and

\[
E_c = 0.0736\omega^{1.51}(f'c)^{0.3}
\]

(3.13)

where in the above equations \( f'c \) and \( E_c \) is in psi, and \( \omega \) is in lb/ft³

\[
\epsilon' = (4.88f'c + 168) \times 10^{-5}
\]

(3.14)
where $f'c$ is in ksi.

For the tension stiffening stress-strain curve and due to the unreasonable mesh sensitivity that can result from using the regular stress-strain curve, especially when there is no reinforcement in significant regions, Hillerborg [51] fracture energy model was implemented in Abaqus. Hillerborg [51] defined the fracture energy $GF$ that required to open a unit area of crack as a material parameter using brittle cracking concept. By using Hillerborg’s fracture energy model, concrete tension stiffening behavior can be defined by stress-displacement (crack width) curve instead of the stress-strain curve. The concrete tension behavior consists of two parts, a linear part up to the maximum tensile strength which is a linear stress-strain relationship followed by a nonlinear descending stress-crack width relationship part that represent the tension softening. Hordijk [54] proposed a complex crack softening relationship for normal and lightweight concrete which will be used to define the concrete tension behavior in Abaqus. Hordijk [54] formula is shown below

$$
\frac{\sigma}{f_t} = \{1 + (c_1 \frac{w}{w_c})^3\} \exp(-c_2 \frac{w}{w_c}) - \frac{w}{w_c} (1 + c_1^3) \exp(-c_2)
$$

where

$c_1=3$, $c_2=6.93$ for normal weight concrete, and $w_c$ is the critical crack width opening which equal to $(5.14 \frac{GF}{f_t})$.

The tension damage factor was determined according to Birtel and Mark [15] by considering the plastic strain to be equal to 10% of the inelastic strain in tension.

$$
d_c = 1 - \frac{\sigma_c E_c^{-1}}{\varepsilon_c^p (1/h_c - 1) + \sigma_c E_c^{-1}}
$$
\[d_t = 1 - \frac{\sigma_t E_c^{-1}}{\varepsilon_t^{pl} (\frac{1}{\nu_t} - 1) + \sigma_t E_c^{-1}}\]  \hspace{1cm} (3.17)

where

\(b_c\) and \(b_t\) is the plastic strain percent of the inelastic strain in compression and tension, respectively.

### 3.5 Soil Material Model

Mohr-Coulomb plasticity model was used for the soil components in this research. It is important to mention that Mohr-Coulomb plasticity model can not capture the post peak behavior because the material model do not account for softening due to soil dilatancy and de-bonding. Figure (3.6) illustrates Mohr-Coulomb yield model and Mohr-Coulomb tension cut-off surfaces. Mohr-Coulomb plasticity model assumes that yield happen when the material’s shear stress equal to a value that depends linearly on the normal stress in the same plane. The yield line in Mohr-Coulomb model is the best line that touches all Mohr’s circles that have been plotted for several states of stress (in the minimum and maximum principles stress plane). Another criteria that is available in Mohr-Coulomb plasticity model is the tension cutoff which is responsible of material hardening and/or softening and soil tension cracking. The range of the friction angle is \(0 < \phi < 90°\).
throughout this research, in the finite elements models (FEM), all the concrete material was modeled using concrete damage plasticity model with parameters that correspond to the material properties specified for the component, subsystem, or system under study. In addition, eight node reduced integration brick element (C3D8R) was used to model the concrete components due of its computational time efficiency and to avoid the numerical problems that results in the full integration elements. element section controls such as hourglass control, distortion control and element deletion were also used in the analyses to reduce the mesh sensitivity, material cracking, and increase the simulation time efficiency.

The steel reinforcement was modeled by using two node 3-D truss element (T3D2) that embedded
inside the concrete elements assuming full bond between the concrete and the steel reinforcement. A multi-linear elastic-plastic stress-strain behavior was adopted for the steel materials with an elastic modulus of 200 GPa and a Poisson’s ratio of 0.29. The steel stress-strain curve consists of an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain. The stress and strain values of the steel material were taken according to Caltrans [20]. No damage or failure criteria were considered for the steel reinforcement.

The Mohr-Coulomb plasticity model was used to define the material behavior of the soil components in this study. A soil friction angle and cohesive yield stress that correspond to the experimental test data was used in Mohr-Coulomb plasticity model. Eight node brick element (C3D8) was used to model the soil components though this research. The tension cut-off was not considered in Mohr-Coulomb model due to lack of soil information.

Material and geometric nonlinearities was considered in the analysis which performed using Dynamic (ABAQUS Explicit) procedure with double precision and low strain rate, which require a very small time step increment, in order to overcome the numerical and convergence problems due to materials nonlinearity and cracking.

3.7 ABAQUS Software Verification

3.7.1 CDP model Verification

The concrete damage plasticity model (CDP) will be used in all the finite element models in this research. In order to verify ABAQUS results on the element level and the CDP model results’ accuracy, patch tests were performed using one element model for the concrete material under monotonic and cyclic loading. These verification results is shown in figure (3.7). The concrete material properties were concrete compressive strength equal to 25 MPa, \( \nu = 0.19 \), and elastic mod-
ulus equal to 24.3 GPa. The concrete material constitutive model assumptions that explained in section (3.4) were used to determine the tension and compression material behavior. Default parameters of the CDP model were used in the verification models.
Figure 3.7: Concrete Damage Plasticity Verification Results
3.7.2 Reinforced Concrete Member Verification

To confirm the CDP model performance in the structural level, a 6m simply supported beam that designed by the author was modeled in ABAQUS and OpenSees to investigate the ability of these packages to predict the correct beam capacity and behavior. ABAQUS beam model includes 17280 brick elements (C3D8R) for the concrete and 635 truss elements (T3D2) for the reinforcing steel. Dilation angle equal to 25° with default values for the other concrete damage plasticity model were used in the model. The steel reinforcement material was represented using bilinear elastic perfectly plastic material with yield strength equal to 460 MPa. The concrete compression strength and Poisson’s ratio were assumed to be 25MPa and 0.3, respectively. Sixteen displacement stiffness based elements were used in OpenSees model with concrete material (concrete02) for the concrete and steel material (steel02) for the steel reinforcement. The loading procedure for both models was represented by applying a displacement control load up to 70mm mid-span deflection. Figure (3.8) illustrate the beam geometry and load-deflection curve obtained by OpenSees and ABAQUS for the beam under study. It can be seen here that both packages provide a very close results and good estimation to the beam capacity and behavior.
3.8 OpenSees Finite Element Package

In OpenSees, a structural component can be modeled with a flexibility based, distributed or lumped plasticity, fiber beam-column element. A fiber beam column element is a line element with several integration points that have an assigned section which in turn used to determine the moment-curvature along the element. The flexibility based method uses the inverted flexibility matrix to obtain the element stiffness matrix. The flexibility-based assumption assumes both the moment distribution along the element and the curvature at the integration points.

The response of the element is then determined by using the weighted integration of the section
responses Taucer et al. [140]. The classical stiffness method is what usually used in a regular non-linear structural analysis program. The stiffness method or displacement method uses the assumed element displacement as the main unknown and uses the displacement interpolation functions to find an approximation for the section deformations. The section resisting forces that correspond to the section formation will be obtained in the next step by using the section force-deformation relation. Weighted integration of the stiffness and section resisting forces along the element yields the element forces. The calculated element forces are assembled to give the structural resisting forces which then compared with the external forces to find the unbalance forces. This unbalance forces will be applied to the structure incrementally and apply the same procedure above till the difference between the external and internal forces satisfied the tolerance criteria.

In the fiber beam column element, this procedure is the same except two things: (a) the element stiffness matrix is derived by inverting the flexibility matrix, and (b) the element resisting forces is determined by assuming the section forces and using nonlinear force-deformation function to approximate the section deformations. The obtained deformations are used to compute the corresponding element deformations. Thus, element forces can be computed using such deformations. The procedure of employing the assumed displacements or forces to find the resisting forces is called the state determination. A summary of the nonlinear fiber beam column element state determination will be presented in the next steps.

(1) The structural state determination, in which the structure external load will be imposed as a sequence of incremental load steps. The internal forces will be computed by performing nonlinear solution algorithm to compare the resulting internal forces with the external forces. The unbalanced forces will be incremented and applied to the structure till tolerance accomplishment. The structural displacements that correspond to the applied forces are then determined.

(2) The element state determination, in which the element resisting forces are obtained from the element deformation. This element level is performed inside the structural level and with each
iteration of the solution algorithm in order to find the element resisting forces with each load step. Force interpolation function and element deformations will be incorporated to provide the section forces and deformations for the Section state determination.

(3) Section state determination, in which the section resisting forces and stiffness matrix are evaluated by integrating the stresses and the tangent modulus over the section cross-sectional area. Subsequently section resisting forces and stiffness matrix employ to yield the section deformation and flexibility matrix respectively. Afterward, the unbalanced section forces and residual section deformations are computed and used to evaluate the element deformation. Figure (3.9) show the flow chart of the element state determinations.
Figure 3.9: Flow chart of element state determination (Tauser et al. [140])
3.9 Bridge Bearing Pad

3.9.1 Introduction

In bridges, several components are connected to each other. Superstructures, column bents, and abutments is the main components of a typical bridge. Bearing pads is the connecting element between these components, and it usually connects the superstructure with abutments and piers. It is also used to mitigate the movements and rotations in the superstructure due to earthquake, moving loads, temperature changes, concrete shrinkage, and creep. There are several types of the bearing pads such as reinforced and non-reinforced elastomeric bearings, laminated elastomeric bearings, sliding bearings, and the POT-shaped bearings. The reinforced elastomeric bearings is the most common bearings that used in bridges which consist of steel reinforcing layers and rubber like materials (elastomers). The movements and rotations are accommodated by the rubber like material (elastomers) which has a flexible resistance under an axial and shear stresses and a stiff resistance against the axial deformation (incompressibility). The bearing pads usually located between two rigid surfaces, which represent the girder surface and the abutment surface or between the girder surface and the bent cap surface. This type of connection can be modeled either as a perfect bond (fixed) or frictional bond, which in turn leads to fixed-fixed, friction-fixed, and friction-friction bonds type of connections. On the other side, bearing pads can be analyzed in two ways, approximate and rigorous numerical analyses.

Early researcher assumed that rubber like materials (elastomeric) are compressible and linearly elastic. Gent and Lindley [41] assumed that the rubber is linear elastic compressible material and the total rubber displacement is the summation of a pure homogeneous deformation of the unbounded block and the subsequent lateral deformation that required to return the end section to the bonded condition. Similar approach was used by Gent and Meinecke [42] to find an approximate
theoretical solution for rubber blocks under low compression (small strain). That solution was extended later to include the bending and shear in elliptical and rectangular cross sectional blocks. Holownia [53] extended Gent and Meinecke [42] solution which considered a small amount of strain (less than 5%) to a high amount of strain (more than 10%) and he modified their equation to include the high strain cases.

As a next stage, the need for the damage, failure, and design criteria and recommendation for the bearings arises due to the isolation that these material provide and the increasing use for the bearings in the structural applications. Roeder and Stanton [113] proposed several provisions for controlling fatigue, delamination yield and rupture in of the reinforcement, stability, and excessive rotation. The authors concluded that these proposed provisions should be taken in account in design of bridges and it will result in substantial increase in the load capacity and height limitation in the elastomeric bridge bearing. Yeoh et al. [147] extended the simple, approximate two dimensional analyses of Gent and Lindley [41] to three dimensional analyses for more complex geometries such as long strip, cylindrical disk, annular block, and rectangular blocks.

With the beginning of the twenty century and with the increasing use of the elastomeric bearings in bridges construction. Key parameters such as bearings shape, type, and material and geometrical nonlinearities that affect the bearings response under static and dynamic loading conditions is the resulting essential study area. Nguyen and Tassoulas [95] extended the early two dimensional plain strain analyses performed by Hamzeh et al. [47] of the bearing to a nonlinear three dimensional analyses taking in account the geometric, material, and contact nonlinearities. They considered the large rubber deformation by means of hyperelastic constitutive model and the frictional contact at the pad-girder and pad-pier or pad-abutment interface by considering the basis of regularized Coulomb model. There models included compression loading in the direction through the bearings thickness followed by shear loading in various lateral direction for square and rectangular laminated elastomeric bearings. They concluded that the bearings stiffness reduction under 50%
shear is negligible. They also found that applying shear to rectangular bearing pads will reduce Mises equivalence maximum stress in the steel shims by 42%.

3.9.2 Finite element model for Elastomeric Bearing Pad

In this section, two finite elements models of the elastomeric bearing pads that was tested by Hamzeh et al. [47] will be presented. Two types of finite elements models are used to model the bearing pad, first is a simple model using OpenSees [102], and the second is a three dimensional continuum finite element model using ABAQUS [5]. The results were then compared to the experimental and the analytical results of Hamzeh et al. [47]. The bearing pad dimension that is used in the models is shown in figure (3.10).

Figure 3.10: Bearing Pad Dimension
3.9.2.1 Abaqus Finite element models

Two models of the bearing pad was created in ABAQUS. The first one was a two dimensional plain strain model and the second one is a three dimensional continuum model. To eliminate the plate flexibility on the buckling behavior, the top and bottom layer of the rubber was connected to an analytical rigid surface with a reference point in the middle which is used to apply the vertical load and the horizontal displacement. By connecting the rubber layers mentioned above to the analytical rigid we assumed that the end plates of the rubber are undeformable which helps to eliminate the plate flexibility. This analytical rigid is an absolute rigid that represent the abutment and the girder or the piers in real bridges. Rough friction was assumed between the top surface of the bearing pad and the top analytical rigid surface while the bottom analytical rigid was tied to the bottom surface of the bearing pad by using a tie constrain available in ABAQUS. Preface bond was assumed between the steel shims and the rubber and was also modeled by using the tie constrain. More details about the two and three dimensional mesh is available in table (3.1).

Table 3.1: Numerical models details

<table>
<thead>
<tr>
<th>Model</th>
<th>Steel layer thickness (in)</th>
<th>Rubber layer thickness (in)</th>
<th>Total rubber thickness (in)</th>
<th>Total bearing thickness (in)</th>
<th>Number of rubber (steel) layers</th>
<th>Total number of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>2D Model</td>
<td>0.1046</td>
<td>0.4375</td>
<td>1.3125</td>
<td>1.5217</td>
<td>8(4)</td>
<td>1024</td>
</tr>
<tr>
<td>3D Model</td>
<td>0.1046</td>
<td>0.4375</td>
<td>1.3125</td>
<td>1.5217</td>
<td>8(4)</td>
<td>1024</td>
</tr>
</tbody>
</table>
3.9.2.1.1 Abaqus 2D Finite element models

The two dimensional plain strain model was modeled with a four node bilinear plane strain quadrilateral, hybrid, constant pressure element (CEP4H) for the rubber and a four node bilinear plan strain element (CPE4) for the steel shims. The plan strain element (CEP4H) use a hybrid formulation in order to be used to represent the hyperelastic incompressible rubber like material like elastomeric rubber. The mesh consists of 8 elements layers for each rubber layer and four elements layers for each steel shim. This mesh was selected in order to exclude the artificial softening of the model due to the increase in the elements number (Kelly and Takirov [64]). Figure (3.11) shows the two dimensional finite element mesh of the bearing pad.

![Two dimension model of the bearing pad](image)

Figure 3.11: Two dimension model of the bearing pad

3.9.2.1.2 Abaqus 3D Finite element models

ABAQUS three dimensional model used eight node linear brick, hybrid with constant pressure element (C3D8H) for the rubber material and an eight node brick element (C3D8) for the steel shims. The rubber elements use the hybrid formulation same as the two dimensional element mentioned above. In order to compare the experimental results and the analytical results of Hamzeh et al. [47], one inch strip width was taken for the three dimensional model. The mesh through the
bearing thickness is similar to the one used in the two dimension model except that the number of elements layers through the width equal 24 to exclude the artificial softening of the model due to the increase in the elements number [64]. Figure (3.12) shows the three dimensional finite element mesh of the bearing pad.

![Figure 3.12: Three dimension model of the bearing pad](image)

3.9.2.2 Material Properties

The rubber was represented by using the theory of hyperelasticity. Several hyperelastic material models are available in Abaqus. These material models can be used for fully incompressible or nearly incompressible behavior of rubber like martial with assumptions that the material is elastic, isotropic, and fully or almost incompressible material. Another assumption is that the geometric nonlinearity is included in the simulation. Yeoh [146] strain energy potential was adopted in the analysis. This model was also used by Hamzeh et al. [47]. Yeoh [146] strain energy potential can
be written in term of the first deviatoric strain invariant $I_1$ of the Cauchy deformation tensor

$$U = C_{10}(\tilde{I}_1 - 3) + C_{20}(\tilde{I}_1 - 3)^2 + C_{30}(\tilde{I}_1 - 3)^3$$

$$+ \frac{1}{D_1}(J^{el} - 1)^2 + \frac{1}{D_2}(J^{el} - 1)^4 + \frac{1}{D_3}(J^{el} - 1)^6 \quad (3.18)$$

where $C_{10}, C_{20}, C_{30}, D_1, D_2, D_3$ are material parameters and $J^{el}$ is the elastic volume ration.

The values of Yeoh [146] strain energy potential parameters are used according to Nguyen and Tassoulas [95] and shown in table (3.2). The steel material is assumed as a bilinear elastic-plastic material with Young’s modulus of 29000 ksi, Poisson’s ratio equal to 0.29, uniaxial yield stress of 40 ksi, and tangent modulus beyond initial yielding of 145 ksi.

<table>
<thead>
<tr>
<th>Model</th>
<th>$C_{10}$ (psi)</th>
<th>$C_{20}$ (psi)</th>
<th>$C_{30}$ (psi)</th>
<th>$D_1$</th>
<th>$D_2$</th>
<th>$D_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2D Model</td>
<td>45.51864</td>
<td>3.09177</td>
<td>0.10048</td>
<td>0.0001</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>3D Model</td>
<td>45.51864</td>
<td>3.09177</td>
<td>0.10048</td>
<td>0.0001</td>
<td>0.0001</td>
<td>0.0001</td>
</tr>
</tbody>
</table>

3.9.2.3 Loading and Boundary Conditions

As mentioned, two analytical rigid were used to represent the bridge girder and abutment or piers. The reference point of the bottom rigid was fixed while the top reference point was constrained in all direction except the direction through the bearing thickness and the transverse direction.

The bearing pad was partitioned in two three parts, two edge parts which represent 40 % of the
total length and one intermediate part that equal to the remaining length. The bottom surface of the intermediate part were tied to the rigid surfaces while the edge parts were connected to the rigid surfaces using the frictionless interaction available in Abaqus. The intermediate top surface of the bearing was connected to the top rigid using friction interaction with a friction coefficient equal to 0.3. This was done in order to capture the roll-over of the front and back edges of the bearing.

The bearing pad was loaded in two steps. First, compression load was applied through the bearing thickness with average stress of 500 psi which is the same applied compression load that considered by Hamzeh et al. [47] in the two dimensional analysis. Then, lateral displacement was applied to the top reference point with an amount equal to 0.875 inch which equal to one-half the total rubber thickness. Figure (3.13) shows the boundary condition and the applied loads on the bearing pad.

![Figure 3.13: Bearing pad Boundary Condition and Loading](image)

3.9.3 OpenSees finite element model

Two dimensional finite element model was created in OpenSees using one 2D elastomeric bearing plasticity element. This bearing element designed to have unidirectional 2D or coupled 3D plasticity properties for the shear deformation and a uniaxial material properties that give it the force deformation ability in 2D or 3D. The element is loaded with compression load through the element thickness followed by shearing the element with a displacement equal to one-half the total rubber

59
thickness. Figure (3.14) shows OpenSees elastomeric bearing plasticity element configuration and material model.

3.9.3.1 Material Properties

The properties of the elastomeric bearing pad element in OpenSees consists of axial and shear deformation ability. The axial deformation ability can be provided by specifying a uniaxial material properties in two or three dimension. The shear deformation capability can be provided by specifying a unidirectional 2D or coupled 3D plasticity properties. A summary of the input parameters for this element is shown in table (3.3). The initial shear stiffness of the bearing that used in the OpenSees model can be calculate using:

\[ K_{shear} = \frac{\Delta \gamma G_n}{T/2} = \alpha \frac{\Delta G_n}{T} \] (3.19)
where $\gamma = 0.5$ and $\alpha$ is a correction factor that account for the bearing edges effects.

The correction factor is an empirical factor that can be obtained by testing the bearing material and it has found that it equal to 0.8-0.9 for three shim pad, 0.95 for six shim pad, and 1 for plane glued pad.

Table 3.3: OpenSees material parameters

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Definition</th>
<th>Used Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m$</td>
<td>Element mass</td>
<td>0.0</td>
</tr>
<tr>
<td>$f_y$</td>
<td>Yield strength</td>
<td>40 ksi</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modulus of elasticity</td>
<td>29000 ksi</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus (Hamzeh et al. [47])</td>
<td>100 psi</td>
</tr>
<tr>
<td>$K_{init}$</td>
<td>Initial elastic stiffness in local direction</td>
<td>0.39 kip/in</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Exponent of nonlinear hardening component</td>
<td>3</td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>Post yield stiffness ratio of linear hardening component</td>
<td>0.01</td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>Post yield stiffness ratio of nonlinear hardening component</td>
<td>0.0</td>
</tr>
<tr>
<td>$sD_{ratio}$</td>
<td>Shear distance from iNode as a fraction of the element length (default = 0.5)</td>
<td>0.5L</td>
</tr>
</tbody>
</table>

3.9.3.2 Loading and Boundary Conditions

Since the elastomeric bearing element in OpenSees is a two dimensional, two node element with three degree of freedom in each node, the boundary conditions consisted of fixing the bottom node and applying the compression load and the lateral displacement at the top node of the element. The applied compression load was 0.5 kip and the lateral displacement was 0.875 inch.
3.9.4 Analyses Results

The analytical and experimental results from Hamzeh et al. [47] have been compared with the finite element models created in OpenSees and ABAQUS three and two dimensional models. Good agreements have been obtained from the finite elements models comparing with the experimental results. The three dimensional ABAQUS model has better results than the plain strain especially in the final steps of the lateral loading where the difference is less than 5%. The OpenSees model has a very good agreement with the experimental results especially when using the correction factor that account for the bearing edges effects equal to (0.8). However, this factor is an experimentally obtained factor which requires experienced assumption due its sensitivity to the bearing’s material and geometry.

![Elastomeric Bearing load-horizontal displacement curve](image)

Figure 3.15: Elastomeric Bearing load-horizontal displacement curve
3.10 Reinforced Concrete Bridge Column

3.10.1 Introduction

Bridge columns is a primary bridge components that have substantial effect on bridges behavior. In fact, columns may be considered the most controlling elements that define bridges’ limit states.

Caltrans [20] consider columns elastic and plastic displacement as an essential part of the bridge global displacement demand. Several bridge components designed depending on columns shear and moment capacity. Superstructure and foundations design specifications recommend that columns should be well beyond its elastic capacity before they reach their nominal strength. Shear keys strength on the other hand should equal to the sum of all columns strength in the weaker frame of the bridge. Transferring the loads between the superstructure and the substructure and maintaining the bridge seismic safety are some of the bridge piers duties and bridge failures due to insufficient capacity of the piers is one of the major research areas that has been studied by many researchers. Piers can undergo several combinations of stress states and loads such as axial-shear, axial-flexure, axial-shear-flexure, axial-torsion, and axial-flexure-shear-torsion.

After investigating columns response under static and dynamic loading conditions, it has been well known that the confinement effect is the most important factor in reinforced concrete columns. Several stress-strain relationship for confined and unconfined concrete were suggested by many researchers over the years. Mander [77] administrated tests on 15 circular columns with spiral reinforcement, 16 rectangular walls and five square columns with rectilinear hoops to verify the results of the proposed generalized stress-strain model of plain or confined concrete under dynamic cyclic with axial compression loading. The author concluded that the proposed relation provide a very good description of the stress-strain performance of the tested column in addition to its capability to predict the response of circular, square, and rectangular columns that have either equal or unequal confinement in each transverse direction.

Damage index, failure modes, and fracture mechanisms represent the next stage research area especially with the ascending importance of bridges seismic design criteria. Ranf et al. [108] tested six circular reinforced concrete columns to investigate the effect of cyclic loading on the damage in lightly reinforced bridge columns. They found that the cumulative plastic damage model is the best method in describing the cyclic load effect. Kim et al. [66] used the damage index to assess the performance of a reinforced concrete bridge column by testing fourteen circular column and comparing the results with the results of a computer program that they developed for the same purpose. The conclusion was that the proposed assessment procedure provide good results and should be used to evaluate the actual bridge columns performance. Bimschas et al. [13, 14] developed a mechanical model to describe the complete distribution of inelastic flexural deformation along the member at all loading levels and compared the results with a large scale reinforced concrete piers tests for verification purposes. The authors believe that the proposed model is promising and it can be easily implemented in a software code or framework and it can be also used to develop the existing expressions to estimate the plastic hinge length. The incapability on the simplified models to provide damage prediction and failure mode and/or mechanism is a
well known substantial deficiency in such models. Yet, for simplicity reasons, studies constantly utilize such models in bridges analyses losing the accurate load path and failure modes. The detailed continuum finite element model, on the other hand, characterized by its ability to provide several damage and failure mode indications.

3.10.2 Column Finite Element Model

The column specimen 407 that was tested by Lehman and Moehle [69] was modeled in ABAQUS and OpenSees. Details of the column under study and its reinforcement configurations is shown in figure (3.16). Further details are available in Lehman and Moehle [69]. Due to the extremely extensive running time that ABAQUS simulation requires and the capability of the monotonic loading condition to provide a good representation of the columns behavior, the column model was loaded with a constant axial load followed by a monotonic lateral displacement history up to experimental maximum lateral displacement of 5 inch. The obtained results from the models were then compared with the experimental results to verify the materials models, the finite element model setting, and to investigate the response differences between the models.
3.10.2.1 Abaqus Finite Elements Model

The concrete and steel element’s types, material constitutive relationships, material models and parameters, and analysis procedures described in sections (3.4 and 3.6) were used in the column’s FEM.
3.10.2.1.1 Materials Data and Parameters

The column concrete materials were divided into two parts, a concrete core and concrete cover. The concrete material properties were taken according to Lehman and Moehle [69]. Unconfined concrete material was used for the concrete cover while the uniaxial stress-strain curve proposed by Mander et al. [78] for confined concrete was used to represent the concrete core behavior in the...
column. Figure (3.18) illustrates the stress strain curve for the confined and unconfined concrete that used to model the column in ABAQUS. The concrete crushing failure in compression was modeled using the concrete compression damage criteria as shown in figure (3.18). Hordijk [54] crack softening relationship for normal weight concrete was used to define the concrete tension behavior in ABAQUS. Figure (3.19) illustrates the crack softening curve and the tension damage-crack width defined in ABAQUS.

![Figure 3.18: Concrete compression stress-strain curves for column 407](image)

Figure 3.18: Concrete compression stress-strain curves for column 407
Default value for the concrete damage plasticity model, except the Dilation angle $\psi$, was used in the ABAQUS model. Table (3.4) provide the geometry, material properties, reinforcement details, and material model parameters for the columns under consideration.
The longitudinal reinforcement material has a yield strain 0.0025 in/in and stress 70 ksi. The yield plateau continued from the yield strain to a strain equal to 0.02 in/in (Lehman and Moehle [69]). The fracture strain and the ultimate strength of the longitudinal reinforcement was 0.2 in/in and 91 ksi, respectively. For the transverse reinforcement, the yield strength, yield strain, ultimate strength, and fracture strain was 0.0035 in/in, 88 ksi, 98 ksi, and 0.07 in/in, respectively.
3.10.2.1.2 Loading, Boundary condition, and Solution Algorithm

The experimental load of the column consists of constant axial load that equal to 7% of the cross sectional capacity \( A_g f'c \) followed be a monotonic lateral displacement applied as a lateral displacement history up to a maximum lateral displacement of 5 inch. The axial load was applied by applying an axial load to a reference point at a rigid body that created and tied to top of the column using tie constrain. Same procedure was applied in the monotonic lateral displacement case, the lateral displacement was applied to the reference point of a rigid body tied to the sides of the loading block. The boundary conditions characterized by fixing the base of the footing and constraining the vertical degree of freedom of a line along the width of the footing with a distance of 1 foot from the footing longitudinal edges representing the experimental boundary conditions.

3.10.2.2 OpenSees Finite Elements Model

The reinforced concrete column was also modeled using the open source finite element package OpenSees. In this model, the column was divided into small fibers in the width and depth directions and each fiber has its material behavior and model. The column under study was discretized into radial fibers with 22 radial core divisions and two radial cover division. The angular divisions were 22 angular divisions. The total length of the columns will be discretized into four elements with 5 Gaussian integration points a long each element. The footing of the column was modeled as two beam-column elements in the longitudinal direction with 48 and 24 fiber divisions in the width and depth directions, respectively. Figure (3.20) shows OpenSees model of the column and the fiber discretization.
3.10.2.2.1 Steel Model

Several steel constitutive models available in OpenSees like bilinear model, Giuffre-Menegotto-Pinto steel material model [140], and reinforcing steel model [91]. Giuffre-Menegotto-Pinto was used to model the reinforcement steel in the column models. This uniaxial steel material model can take account for Bauschinger effect and isotropic hardening but it does not account for degrading in strength and stiffness due to bar buckling and cyclic degradation.
Concrete material with linear tension softening available in OpenSees was used to model the concrete material in column 407 model. Mander et al. [78] proposed a concrete model that accounts for the confinement effect on the maximum concrete compression strength which should be determined as the following:

\[
f_{cc}' = f_c'(-1.254 - 2\frac{f_t'}{f_{co}} + 2.254\sqrt{1 + 7.94\frac{f_t'}{f_{co}}})
\]

(3.20)

\[
\varepsilon_{cc} = \varepsilon_{co}[1 + 5\left(\frac{f_{cc}'}{f_{co}} - 1\right)]
\]

(3.21)

\[
f_t' = \frac{1}{2}k_c f_{yh}\rho_s
\]

(3.22)
where \( f'_{cc} \) is the confined concrete compression strength, \( \varepsilon_{cc} \) is the confined concrete compression strain, \( f'_{co} \) is the unconfined concrete compression strength, \( \varepsilon_{co} \) is the unconfined concrete compression strain, \( d_c \) is the core diameter, \( \rho_{cc} \) ratio of the longitudinal reinforcement area to the core area, and \( f_{yh}, d_s, A_{sp}, s', s \) is the yield stress, diameter, area, clear vertical spacing, and center to center spacing of the spiral reinforcement. Mander et al. [78] model was used in OpenSees model to define the confined concrete core while the concrete cover was defined using the concrete linear tension softening (concrete02) material model. Figure (3.22) shows the stress-strain relationship for the concrete in tension and compression. For the concrete tension strength, linear tension stress-strain relationship was assumed up to a tensile strength of 420 psi (Lehman and Moehle [69]) followed by a linear degradation to zero tensile strength. The other material properties of the concrete and steel material were used according to the experimental values available in Lehman and Moehle [69].
3.10.2.2.3 Loading and Boundary Conditions

The boundary conditions characterized as roller support at the footing exterior nodes and hinge support at the column base node. The loading consists of a constant axial load of 147 kip followed by a reversal lateral displacement that equal to 1, 1.5, 2, 3, and 5 times the displacement ductility [69]. Both loads was applied at the column tip with three cycles for each lateral displacement value. The analyses were performed as a static load for the axial load followed by a displacement control push over analysis for the cyclic load. Figure (3.23) show the displacement history that was used in the OpenSees pushover analysis.
3.10.2.3 Column finite element Results

It was noticed that ABAQUS model computational time increases significantly with increasing the loading rate due to the high nonlinearity that results from material cracking and cracks opening and closing. On the other hand, reducing the load rate will also leads to long computational time. The displacement history for column 407 that was used in the experimental test consists of five cycles with lateral displacement equal to 1, 1.5, 2, 3, and 5 times the displacement ductility. It was found from several trials that the optimum loading rate is approximately 0.1 in./sec which requires extremely extensive running time to simulate the total displacement history.

As reported by Ranf [109] that damage level (spalling and yielding) are not dependent on the loading history but on the maximum displacement that the structure experience. Due the aforementioned reasons, the column was loaded monotonically using displacement control loading with loading rate equal to 0.1 in/sec to simulate quasi-static loading. Figure (3.24) illustrate ABAQUS and OpenSees finite elements results comparing to the experimental results. Both ABAQUS and
OpenSees provide good agreement comparing with the experimental test results. However, by comparing the displacement components obtained in the experimental test with OpenSees results, it can be noticed that OpenSees model was able to capture the displacement bending component without capturing the other displacement components that are shear and slip displacement. In addition, it was incapable of capturing concrete and reinforcement failures such as spalling and reinforcement buckling and fracture. OpenSees results indicate that such simplified modeling for bridge components provide good prediction for the global behavior but can not capture the local behavior and lack the ability to simulate damage and tridirectional behavior.
Figure 3.24: Column 407 Analyses Results

(a) Column 407 OpenSees Results

(b) Column 407 ABAQUS Results
ABAQUS on the other hand, despite its extensive computational time and convergence difficulties that results due to material nonlinearity can capture localized damage like spalling and cracking in addition to the local behavior and tridirectional effects. Figures (3.25 and 3.26) show concrete tension and compression damage. Figure (3.27) shows ABAQUS predicted compressive strain in the concrete cover comparing to CALTRANS 2013 recommended spalling strain. According to Lehman and Moehle [69], the concrete spalling area height at 2 inch displacement (displacement ductility equal 3) was 3 inch and at 3 inch displacement (displacement ductility equal 4) was 10 inch. By comparing the results in figure (3.27) with the observed experimental results, it can be observed that the FEM provided good prediction for the concrete spalling and other column’s local damage.

Figure 3.25: Column Compression Damage
Figure 3.26: Column Tension Damage

Figure 3.27: Predicted compression strain on the column cover
3.11 Bridge Abutment’s Backfill

3.11.1 Introduction

Each bridge have an abutment at each end. Abutments provide inlet and outlet for the traffic to and from the bridge. In case of the bridge itself, abutments provide force resistance of the bridge loads and a supporting system against ground motions. Classical abutment components are: a) Back wall that limits the embankment soil borders and isolate the bridge deck from the embankment soil. b) Stem wall that carry the superstructure load and transfer it to the foundation system. c) Foundation or pile system that transfer the load to the ground. d) Two wing walls that provide partial resistance to superstructure pushing forces and laterally restrain the back fill material. e) Shear keys which laterally support superstructure forces and motility.

As a design point of view, abutment back walls generally design to split from the stem wall and pass on deck compression forces to the soils back fill on which most design philosophies depend on as the only resisting element to that forces. Stem wall and piles or foundations on the other hand commonly design to resist the compression forces from the deck and deliver it to the ground. Accordingly, back fill-abutment interaction have the foremost role in resisting bridges lateral movement and it may aggravate bridges behavior from elastic to nonlinear behavior when subjected to strong ground motion. Moreover, in case of seismic load, a nonlinear force displacement response of the abutment could consequence from the passive pressure that has been delivered to the abutment back fill. This response is quite complex and despite the fact that it has been studied by many researchers, it is yet not fully understood.

The two major complicated research areas that related to the abutment systems are the effect of structure-abutment-back fill interaction on the bridge response and the nonlinear back fill-abutment relationship. These areas have been experimentally and theoretically studied by various
researchers. Early researchers tried to understand the real structural-back fill soil relation and estimate the stiffness and capacity of abutment system. Siddharthan et al. [135] introduced a simple methodology to evaluate the nonlinear longitudinal and transverse springs stiffness of abutment that connected to a pile foundation. The presented approach considered abutment dimension, nonlinear pile-soil interaction, superstructure load, and soil behavior. The result that have validated with other experimental results showed that the pile displacement and fixity conditions have a significant effect on the abutment stiffness in the longitudinal directions. Bozorgzadeh et al. [18] examined the structure-back fill interaction effect on the abutment stiffness and capacity in the longitudinal direction. Effect of back fill soil type, back fill height, wall vertical movement, and pre-existing cut slope were studied. It was found that the soil shear strength and interface friction play a major role in the structure back fill passive resistance.

Shamsabadi et al. [127] introduced a formulations for the mobilized force-displacement capacity and stiffness of embankment-abutment system which take into account interface angle between the abutment and embankment, abutment height, and embankment soil properties. Results of these formulations were verified with a small and full scale experimental results with a good agreement. The aforementioned relations were then used by the same author [129] to develop a model that was in turn implemented in a nonlinear 3D model to study the dynamic performance of bridge abutments. It is concluded that inadequate abutment seat width may cause superstructure unseating or large displacement that leads to excessive columns moments and displacements. It is concluded that inadequate abutment seat width may cause superstructure unseating or large displacement that leads to excessive columns moments and displacements. Latter, Shamsabadi et al. [128] validated both the numerical model and the finite element model that have been developed in the previous papers with two full scale tests and developed the numerical model to predict a simplified force displacement backbone curve for abutments with different back wall heights. Authors concluded that the proposed model provide more accurate backbone curves than Caltrans [20] current approx-
imation, although, this model is restricted to the tested types of soil and can be developed for other soil types.

3.11.2 Abutment Back Fill Model

A continuum three dimensional finite elements model of the full scale abutment back wall cyclic test conducted by Stewart et al. [138] is assembled using ABAQUS and OpenSees. The back wall dimension was 2.6 m height, 4.57 m width, and 0.9 m thick. The natural clayey soil was replaced with the back fill silty sand soil known as sand equivalent 30 (SE30) which is Caltrans [20] standard abutment back fill materials. Plywood panels were constructed on each side of the back fill to represent the wing walls with a distance of 0.3 m from the back wall. The plywood wing walls were covered with two layers of 0.006 in PVC foil to reduce the friction between the back fill and the sidewalls. The back fill slope and extension behind the back wall is shown in figure (3.28). Reinforced concrete with a unit weight of 145 pcf and compression strength of 5.8 ksi was used to construct the back wall. The yield strength of the steel reinforcement was 60 ksi.
3.11.2.1 Abaqus Finite Elements Model

ABAQUS model consisted of 15000 brick elements (C3D8) for the back fill, 7000 elements for the ground soil and the sidewalls, 720 elements for the back wall and 3000 elements (T3D2) for the reinforcement. Figure (3.29) show the finite elements model of the back wall model.
3.11.2.1.1 Material Model

Mohr-Coulomb plasticity model was used for the back fill materials. According to Stewart et al. [138] the friction angle and cohesive yield stress equal to 39.5 degree and 36.7 kPa, respectively. The natural ground soil and the sidewalls were modeled as a rigid, almost incompressible, linear elastic material to insure that the failure surface occurs in the back fill region.

Concrete smeared cracking material model with default values of the failure ratio, shear retention, and tension stiffening parameters were used. The experimentally obtained uniaxial stress-strain data and the load displacement history shown in figure (3.30(a)) were used for the concrete compression response and loading history, respectively. The reinforcement were modeled as embedded
elements inside the concrete elements with a linear elastic material.

### 3.11.2.1.2 Boundary conditions, loading and solution technique

The boundary conditions characterized by fixing all the exterior boundaries of the ground soil and the sidewalls. Backfill bottom surface was tied to the natural ground top surface using tie constrain assuming perfect connection between the two surfaces. Frictionless contact was used to model the contact between the sidewalls and the back fill (Stewart et al. [138]). The friction angle between the back wall and the back fill was taken equal to one third of the soil friction angle while 14 degrees was used as the friction angle between the back wall and the natural ground soil as specified by Stewart et al. [138]. Analytical rigid with a reference point that was tied to the back wall and used to apply the displacement history shown in figure (3.30(b)). Static general (ABAQUS Standard) procedure with unsymmetrical matrix solver and full newton solution technique was employed in order to overcome the numerical difficulties due to shear and tensile failure inside the back fill materials.
3.11.2.2 OpenSees Model

The back wall experimental test was modeled in OpenSees using two elastic beam-column elements for the back wall and three zero length elements (springs) for the back fill. Concrete material were assumed as an elastic concrete without reinforcement. The embankment passive pressure resistance and stiffness were calculate according to Caltrans [20]. The embankment material was modeled as a bilinear material with the embankment initial stiffness of 50 kip/in/ft and passive pressure of 5 ksf. Pushover analyses of the back wall model in the direction perpendicular to the back wall plane were performed using the displacement history shown in figure (3.30(b)). Figure (3.31) shows the OpenSees model of the abutment back fill.
3.11.3 Abutment Back Fill Results

ABAQUS results show a good prediction of the back fill deformed shape and can be used to estimate the failure surface of the soil. Figure (3.32) illustrates the back fill plastic strain envelope while figures (3.34 and 3.35) shows the deformed shape for the back fill obtained from FEM. Based on the strain distribution and deformed shape shown in figures and comparing to the experimental failure surface, there is close agreement with the experimental soil failure surface (similar trend) and surface rupture location. It can be also noticed from the strain concentration and back fill deformation areas that the failure mechanism is within the fill (and not embankment) as recommended by the design codes.
Figure 3.32: Back fill plastic strain ($\sqrt{2/3\varepsilon_r^{pl}} : \varepsilon^{pl}$)

Figure 3.33: Experimental crack location and failure surface (Stewart et al. [138])
Figure (3.36) shows the force-deformation results for the back fill. By comparing the stiffness, strength, and overall response shown in the figure, it can be noted that the FEM provided a good prediction for the back fill nonlinear behavior and estimation for the component resistance. The simplified model overestimated the stiffness and underestimated the strength by 10% and 15%, respectively, relative to the experiment.
Several observation were found from comparing the load deflection curve of the experimental test with Abaqus and OpenSees curves. First, Abaqus continuum model provide better representation of the back fill behavior than OpenSees model which uses elastic-plastic material behavior with stiffness and passive pressure based on Caltrans [20]. Second, in contrast to OpenSees model, ABAQUS model capture the embankment nonlinear response despite the fact that the back fill material model was defined without any damage criteria or tension cutoff. Defining soil tension cutoff will help to capture the hardening and softening response of the soil. The incomplete unloading displacement resulted in higher unloading strength for both models can be referred to the material elastic recovery and contact interaction between the back wall and back fill surfaces. Caltrans [20] provisions underestimate the abutment back fill by about 15%. Better representation of the back fill material should be used in OpenSees model to provide better estimation for the back fill strength.

Figure 3.36: Comparison of the load deflection curve for the abutment back fill
3.12 Bridge Superstructure

3.12.1 Introduction

Worldwide, concrete bridges are one of the very common bridge types. Concrete bridges are mostly constructed using concrete or pre-stressed concrete which are usually formed as in-situ casted or pre-casted T-girder beams, I-beams, or box girders. Concrete girder bridges characterized by their large mass comparing with steel girder bridges. Coupling this large mass with bearing pads flexibility produce large displacements pounding forces in earthquake events. Despite the observed elastic behavior of bridges superstructure which is recommended by most design codes, several failure cases of T-girder beams, I-beams, and box girders were observed due to dead and seismic loading conditions [145, 63, 49]. Such failure cases motivated further studies to investigate the real response of these girders individually and collectively and their effect on the entire bridge response.

Literature lake sufficient information on the performance of bridge girders especially for experimental tests due to the cost, instrumentation experience, space for such tests, and most importantly the availability of test specimens. Song et al. [137] performed a destructive test on a two full-scale in-situ deteriorated reinforced concrete T-girder under cyclic loading up to failure. One of the tested T-girders is a symmetrically loaded girder while the other was unsymmetrically loaded girder. A two dimensional nonlinear finite element analyses for the symmetrically loaded girder was also performed using Reinforced concrete zoning method. The non-symmetrically loaded girder was modeled using a three dimensional model using frame and shell elements. The authors concluded that old bridges retain high load carrying capacity and the modeling procedure that used to model degraded supports significantly effects the predicted stiffness and capacity of the bridges.

Park et al. [103] developed an extended method to determine the exact distortional behavior of
multi-cell box girders. The proposed method uses force equilibrium to break down the eccentric load into flexure, torsional, and distortional loads which in turn used to obtain the distortional, flexural, and torsional behavior. As claimed, the proposed method results in addition to results obtained by using this method in an independent shell analyses have shown good agreement comparing with the conventional shell analysis results. Harries [49] tested two 42 years old pre-stressed concrete box girders, that recovered from Lake View Drive Bridge partial collapse, up to failure due to flexure. The author concluded that the main reason of failure is the pre-stress force loss due to strands corrosion which was indicated from sever transfer cracking of the girders soffit. He also concluded that barrier walls resistance was considered in the exterior girders design which led to failure after the composite action degradation.

3.12.2 T-Girder bridge Model

The first target bridge section that has been tested by Song et al. [137] was modeled in both OpenSees and ABAQUS. The 50-years old target bridge under the experimental study consisted eleven cast in place simply supported spans with three 12 m T-girders for each span. However, following the experimental test and the analytical finite elements analyses performed by Song et al. [137], the t-girder length considered in the finite element model was 7.75 m which represent the length tested and analyzed by the author. The best conditions single T-girder was longitudinally cut from the total bridge span for the symmetric loading destructive bridge test. The reinforcing bars and the concrete cores tests showed that the concrete has an average of 26470 and 35.3 MPa for the initial elastic modulus and compressive strength, respectively. Figure (3.37) shows the T-girder bridge section.
3.12.2.1 Abaqus Finite Elements Model

AB AQUS model consisted of 30000 brick elements for the concrete and 4700 truss element for the reinforcement. Finer mesh was used in the mid span and at the supports to overcome the mesh sensitivity due to stress concentration. Analysis procedure, elements’ section control and types were used according to section (3.6). Figure (3.38) illustrates the finite element model of the bridge T-girder.
Figure 3.38: Three dimensional model of the T-girder bridge
3.12.2.1.1 Abaqus Material Models

The concrete material compression stress-strain relationship, tension stress-crack with relationship, and compression and tension damage factors were obtained following section (3.4). The concrete tensile and compressive uniaxial stress-strain curves are shown in figure (3.39).

![Compression Stress-strain Curve](image1)

![Tension Stress-crack width relationship](image2)

(a) Compression stress-strain Curve  
(b) Tension stress-crack width relationship

Figure 3.39: ABAQUS material models

Default values were used for the concrete damage plasticity parameters that are; \( K = 2/3 \), \( \sigma_{bo}/\sigma_{co} = 1.16 \), eccentricity = 0.1, and viscosity parameter = 0.0. On the other hand, due to lack of the concrete material tests that help finding the dilation angle and due to the T-girder’s high aspect ratio that induces shear controlled behavior which in turn causes brittle response, 54 degrees was used for the dilation angle to provide more ductile behavior and avoid unrealistic premature failure. The steel reinforcement material was modeled as a bilinear elastic-plastic material with 435.8 MPa yield strength and a hardening ratio of 2%. The other properties, which have been assumed, were
\( E_s = 200 \text{ GPa}, \nu = 0.29, \text{ and } \gamma = 7850 \text{ kg/m}^3. \)

### 3.12.2.1.2 Loading, Boundary conditions, and Solution Algorithm

Simply-supported boundary conditions configuration were considered in the finite elements model. This boundary conditions were modeled by creating two rigid bodies for the supports and specifying frictionless interaction to represent the roller boundary and a friction coefficient of 0.4 to represent the hinge boundary conditions. For a realistic test modeling, another rigid body was created to apply the mid span cyclic displacement on the girder. The experimental test was performed by applying a load control procedure with load values of 40, 55, 75, and 108 ton. Although, a displacement control procedure was applied in the finite elements analysis by applying the same deflection that obtained from the experimental test. It should be pointed out here that despite the analysis procedure and the element section controls that have been used in the analysis, and due to the materials’ nonlinearity, shear dominant behavior, and excessive cracking, the analyses using explicit procedure (ABAQUS explicit) required an extensive running time with an extremely small time step. Another important observation is that the finite element model developed by Song et al. [137] considered using a horizontal spring element with a stiffness \( k = 600 \text{ KN/mm} \) at the roller support in order to represent the in-situ boundary conditions and to obtain the reloading and unloading experimental behavior. This stiffness was calculated from calibration with the load-deflection curve initial stiffness that obtained from the service load test of the target bridge. This setup, yet, was not considered in ABAQUS model which may lead to different unloading behavior.

### 3.12.2.2 T-Girder bridge OpenSees Model

OpenSees model for the T-girder was modeled using the flexibility based, distributed plasticity fiber beam-column element. Eight displacement beam-column elements were used to model the
girder with simply-supported boundary conditions and five Gaussian integration points for each element. Figure (3.40) shows OpenSees mode and fiber discretization for the T-girder specimen.

![T-Girder fiber Section and Element Configuration](image)

(a) T-Girder fiber Section  (b) Element Configuration

Figure 3.40: OpenSees T-Girder Model

The uniaxial material with linear tension stiffening (concrete02) was used to model the concrete material assuming the residual compressive strength equal to 20 % of the maximum compressive strength. The steel material was modeled using the uniaxial Giuffre-Menegotto-Pinto steel material available in OpenSees with default values for the parameters that control the transitions from elastic to plastic branches. The concrete part was divided into 750 fiber division for the flange and 450 fiber division for the girder web below the flange. As mentioned earlier, the concrete compressive strength was 35.3 MPa while the tensile strength was determined using $0.625\sqrt{f'_{c}}$ in MPa. The other material properties were taken from the experimental test.

Same ABAQUS loading and boundary conditions were modeled in OpenSees which represented by simply-support boundary conditions and displacement control cyclic loading at the mid-span. Nonlinear pushover analyses was performed using a displacement control protocol and Newton-Raphson algorithm solution technique.
3.12.2.3  T-Girder Analyses Results

The finite elements models of both ABAQUS and OpenSees provided good agreement with the test results as shown in figure (3.41). However, several observations can be deduced by comparing the analyses and experimental test results. First, despite the accurate strength and initial stiffness, both analyses results could not capture the aging effect on the girder stiffness and strength which can be seen in the initial load cycle in ABAQUS results and the pre-loading step in OpenSees. In addition, due the fact that the T-girder under study is 50 years old, a pre-loading step was created in OpenSees model to simulate the aging effect and reduce the girder capacity to the experimentally obtained value. Second, due the undefined experimental boundary conditions, the T-girder unloading behavior could not be simulated in ABAQUS model while about 50% of the unloading displacement could be obtained in OpenSees model using vertical and horizontal elastic zero length elements with elastic modulus equal to 700 and 24 KN/mm², respectively. Moreover, a simple hand calculations, which were done by simulating the girder and supports as a group of springs to obtain an approximation for the mentioned springs stiffnesses, indicated that the vertical springs stiffness that can represent the experimental boundary conditions may varies from 25 to 550 KN/mm and the experimentally use supports do not provide full fixity.
Figure 3.41: T-Girder Finite Elements Results
The final observation is the shear controlled failure mode of the T-girder that can be seen in ABAQUS tension damage shown in figure (3.42). It should be mentioned here that, OpenSees model was created using displacement based elements instead of the force based elements due the inadequacy of the latter to provide accurate results for beams with high aspect ratio. This failure mode that shows shear controlled behavior confirm the shortcoming in the one dimensional elements to provide accurate results for members with high aspect ratio. This shortage could be attributed to the elements basic assumptions such as plane section remain plane, the curvature is linear, and stress and strain distribution is linear and parallel to the member axis. This deficiency represent one of the major drawbacks in the 1D elements that limit the spine models capability to predict the actual response in several applications such as deep beams, shear walls, and bridge girders.
Figure 3.43: T-Girder Compression Damage

Figure 3.44: T-Girder Horizontal Stress Contours
3.13 Shear Key Bridge Components

3.13.1 Introduction

The main function of the shear keys is the lateral support of the bridge superstructure under lateral loads condition. Their main duty in earthquake events is to transfer the reaction loads from the superstructures to the abutments which in turn handover it to the ground through the shear forces in the wing-walls and piles. Many references call them sacrificial shear keys due to their assumed sacrificial role and since they are not designed to provide any additional resistance after exceeding the capacity. The only element that is expected to provide full transfer support to the entire length of the bridge superstructure after shear keys failure is the column bents (CALTRANS Bridge Memo to Designers Manual).

Caltrans bridge design specifications Caltrans [20], consider the abutments damage due to major earthquake events is acceptable if it will not cause bridge collapse or superstructure unseating. Moreover, Caltrans [20] limited the transfer input force that used in bridge design as the shear key design capacity in order to control the damage in the abutments and piles. The transverse seismic design force for the shear keys was limited to 75% of the abutment piles shear capacity plus one wing-wall shear capacity.

Two types of shear keys can usually be constructed in abutments, interior and exterior shear keys. The exterior shear keys should be constructed at the superstructure sides to provide lateral support and can work in one direction only while the interior shear keys should be constructed in the stem-wall and within the superstructure width and can work in both directions. However, because the interior shear keys are not accessible for repairs and maintenance, recent design specifications recommended avoiding interior shear keys in bridge design.
Literature is limited in the shear keys seismic behavior. Experimentally, Megally et al. [87] tested six exterior sacrificial shear key specimens to study the seismic behavior of the exterior shear keys. Investigated parameters included back and wing walls contribution, post tensioning of the abutment stem wall, and using different shear key configuration such as construction joints and using flexure key instead of shear key. It was found that Caltrans [20] specifications considerably underestimate the shear keys capacity which lead to overloading the abutments and piles. Bozorgzadeh et al. [17, 18] carried out six exterior sacrificial shear key specimen tests to study the effect of using construction joint between the shear key and abutment’s stem wall on the shear key seismic response. The conclusion emphasized on the importance of considering a smooth construction joint between the shear key and abutment’s stem wall to allow sliding shear failure and prevent abutment and piles overload and damage due to the considerable underestimation of the shear key capacity. Analytically, Goel and Chopra [45] investigated the shear key role on the seismic behavior of regular bridges. It has been found that due to the shear key conservative design (maintained elastic behavior), the seismic demand of a bridge with nonlinear shear key can be limited between the demands of a bridge with elastic shear keys and a bridge with no shear keys.

3.13.2 Shear Key Model

The shear key specimen of series I, test unit 1A that tested by Megally et al. [88] was modeled in Abaqus and OpenSees. This test unit which match an existing Caltrans abutment as-built bridge design details was built without back wall and wing wall and was designed at a scale of 2/5 (Megally et al. [88]). The test specimen series I represented an abutment with two test units, test unit 1A on the right side which consists of shear key and test unit 1B on the left which consists of shear key, back and wing wall. Figure (3.45) shows the reinforcement layout of test specimen series.
Figure 3.45: Shear key reinforcement layout (Megally et al. [88])
3.13.2.1 ABAQUS Finite Elements Models

Only the right half of series test I (unit 1A) was modeled in ABAQUS assuming symmetric boundary condition at the center line of the abutment. Unit 1A consists of a single shear key that connected to the stem wall and foundation block. The concrete part was modeled using 36000 brick elements. Bias control was used to concentrate the element in the cracking region to avoid numerical difficulties due to excessive elements cracking and better crack path prediction. Analysis procedure, elements’ section control and types were used according to section (3.6). Figure (3.46) shows the ABAQUS finite elements model.

Figure 3.46: Abaqus finite element model
3.13.2.2 Material Properties

The material parameters were 2382 Kg/m$^3$, 34.2 MPa, and 0.2 for the concrete unit weight, compression strength, and passion ratio, respectively. The concrete material compression stress-strain relationship, tension stress-crack with relationship, and compression and tension damage factors were obtained following section (3.4). The steel yield strength and ultimate strength were 448 MPa and 676 MPa, respectively. Figure (3.47) shows the compression and tension stress strain relationships that have been used in ABAQUS model.

![Compression Stress vs Strain](image1.png)

(a) Compression stress-strain curve

![Tension Stress vs Crack Width](image2.png)

(b) Tension stress-crack width curve

Figure 3.47: Abaqus Shear key concrete material relationships

3.13.2.3 Loading and boundary conditions

Since half of the abutment was considered in the study, symmetry boundary conditions were applied at the edge that represent the mid of the abutment. The experimental test set up included post
tensioning the foundation block with ten tie down bars at the shear key sides and one central tie down bar at the top of the stem wall each with 667KN tension force.

This setup can insure foundation block fixity which was also assumed in the finite element model. In addition, the vertical displacement in the lines that pass through at the tie down bars was constrained to represent the tie down bars constrain at the foundation block. Experimentally, the lateral load was applied by using a loading arm with a cyclic loading protocol that consists of applying the load in force control for the first two cycles and switching to displacement control for the remaining cycles. In the finite element model, the loading arm was modeled using a rigid body and the loading procedure was changed to a displacement control loading protocol shown in figure (3.48) after calibrating the first two displacement cycles to provide the same applied experimental force.

![Shear key displacement history](image)

Figure 3.48: Shear key displacement history
3.13.3 Shear key OpenSees Model

The second model of the test unit 1A was created in OpenSees using zero length element and a hysteretic material model available in OpenSees. Figure (3.49) illustrate the shear key OpenSees model and the hysteretic material model. The force-displacement relationship of the hysteretic material was obtained using the experimental force-displacement curve for the test unit under study. Same loading procedure that used in ABAQUS model was used in OpenSees which consists of applying a displacement history shown in figure (3.48) using displacement control procedure.

![Figure 3.49: OpenSees Material Model for the Shear key](image)

3.13.4 Shear key analyses Results

The concrete brittle cracking model available in ABAQUS provide better representation for the cases governed by shear or tension failure. However due to the cyclic loading procedure that used in the shear key test, it is preferable to use concrete damage plasticity model due to its better representation of both compression and tension behavior.
Figures (3.50 and 3.51) shows the numerical load-displacement curves compared to the experimental results. The FEM results provided a reasonable prediction for the strength, stiffness degradation, reinforcement yielding, and the overall behavior. The difference in the strength in the initial displacement cycles is due to the displacement jump that occurred in the experimental test, and the use of 20% residual tension strength to avoid the numerical problems that resulted due to severe shear stresses. In addition, because the FEM did not include steel failure or fracture, a yielding plateau in the final displacement cycles was observed unlike the rapid softening in the experimental test. The simplified shear key model provided a good estimation of the shear key strength and overall behavior due to the calibration based on the testing program. However, studies have indicated that the developed formulas cause significant underestimation of the actual shear key capacity and stiffness [101, 45].

![Figure 3.50: Shear key ABAQUS Lateral Load - Horizontal Displacement Curve](image)
Figure (3.52) illustrates the concrete cracking pattern characterized by concrete tension damage at the same loading levels of the experimental test (Level II and III) according to Megally et al. [88]). The tension damage represents the degradation of the elastic stiffness in tension (i.e., damage of 1.0 reflects total loss of tension stiffness). The figure shows very close crack patterns and observable concrete damage to the experiment test.
Figure 3.52: Shear key Crack Pattern

(a) Level II Experimental Damage Megally et al. [86]
(b) Level III Experimental Damage Megally et al. [86]
(c) Level II ABAQUS
(d) Level III ABAQUS
CHAPTER 4: BRIDGE SUBSYSTEMS

4.1 Introduction

One of the common techniques that can be used to study bridges is subdividing it into subsystems. A bridge subsystem is a group of bridge components that are connected to each other, behave as one unit, effect each other behavior, and share the same load path. Example of bridges’ subsystems are abutment subsystem, columns bent subsystem, and superstructure subsystem. Using such techniques help studying such system with high fidelity, predict such system effect on bridges’ behavior, and overcome the computational difficulties that arise in full bridges analysis.

4.2 Abutment Subsystem

Many post-earthquake reconnaissance reports emphasized the drastic effect of abutment behavior, soil structural interaction, and embankment flexibility on the seismic response of the whole bridge system especially under moderate to strong intensity of earthquake ground motion. Seat-type abutments are a common abutment types in highway bridge structures with various functions such as retaining of back fill soil, supporting superstructure and vehicles gravity load, and providing traffic transitions between a bridge and its approaches. Such abutment type consists of back fill and embankment, back wall, wing walls, stem wall, shear keys, and foundation. Typically, in addition to the reinforced concrete bridge components that are designed to perform inelastically during earthquake events, bridges’ abutments are characterized by their multi-nonlinear resources that result from embankment soil, backfill soil, foundation, bearings, stoppers, and seismic joints. To obtain a realistic representation of these elements, a detailed continuum finite elements models or efficiently sophisticated simplified representation of these elements should be employed in their
modeling and analysis procedures. These simplified models should include all the main components and resisting mechanism with an accurate evaluation of their mass, stiffness, strength, and nonlinear hysteretic behavior.

During seismic events, a bridge superstructure moves longitudinally or transversely, and may impact the back wall and shear keys, respectively, inducing deformations and internal forces in the abutment. Several guidelines and specifications require that abutment-soil systems be included in the analytical models as linear springs [1, 9, 20]. Design practices, e.g., California [20] and Washington [12], use bi-linear force-deformation curves, based on select few experimental tests, for the back fill and abutment properties. Specifications typically consider the back wall as a sacrificial element (as a failure mode assumption) that shears off to protect abutment walls and piles from excessive force and/or damage. Accordingly, the longitudinal load path includes the back wall-soil interaction and the passive resistance of the back fill. In the transverse direction, the failure mode assumption is that shear keys are sacrificial elements to protect the joint, walls, and piles from excessive force and/or damage. The energy imparted by superstructure impact should be dissipated by the shear keys, preventing the deck from unseating and helping the abutment components to maintain elastic behavior. In both cases, the capacity of the back wall and shear key are such that they prevent motion of the superstructure under serviceability-level motions.

Despite the aforementioned load path and failure modes assumption that are recommended by design codes and adopted in bridges design and analysis procedures. It was observed experimentally and analytically that abutment components are behaving differently that results in several deficiencies in these procedures which in turn lead misunderstanding to the actual behavior.

California’s Murphys Grade Road (MGR) is a typical reinforced concrete box girder bridge with seat-type abutments and pile foundations. The implementation of the model follows the work of Aviram et al. [10]. First bridge subsystem will be the MGR abutment subsystem which consists of
piles cap, stem wall, wing walls, back wall, shear keys, and back fill and embankment. The MGR abutments was modeled in both, the finite elements program ABAQUS [5] and the finite elements platform OpenSees [85]. The abutment dimensions is illustrated in figure (4.1). Figure (4.2) shows the abutment subsystem ABAQUS model.

![MGR abutment geometrical properties](image)

**Figure 4.1: MGR abutment geometrical properties**

### 4.2.1 ABAQUS Finite Elements Model

The concrete and steel element’s types, material constitutive relationships, material models and parameters, and analysis procedures described in sections (3.4 and 3.6) were used in the abutment subsystem’s FEM. A fine finite elements mesh was used in the stress consternation areas and sudden geometry change that is: the shear keys, backwall, and wing wall-backwall connections. The embankment length was taken equal to 21 m to reduce its effect on the abutment behavior. It should be mentioned here that the longitudinal load path and failure mode assumption adopted in
this study is according to Caltrans [20].

![Figure 4.2: ABAQUS model for the abutment subsystem](image)

Figure (4.3) illustrates the concrete compression stress-strain curve and tension softening curve used in ABAQUS. The concrete compression strength and Poisson’s ratio were equal to 25 MPa and 0.2, respectively. The concrete damage plasticity model parameters adopted for the shear key FEM (section 3.13) were used here with a compressive stress-strain and tension softening relationships that were updated using the material properties adopted in the abutment design process. The steel was modeled an elastic-perfectly-plastic material and yield stress equal to 414 MPa.
For the backfill and embankments materials, Mohr-Coulomb plasticity model available in ABAQUS was used with a material properties according to Stewart et al. [138] as shown in table (4.1). Also, back fill-back wall and back fill-wing walls friction was considered in the analyses using the friction interaction available in ABAQUS with a friction angle equal to 14° according to Stewart et al. [138]. Same soil material parameters and constitutive relationships that were used in the abutment back fill model (3.11) were used in the abutment subsystem study.

Table 4.1: Backfill material parameters

<table>
<thead>
<tr>
<th>$\gamma$ (lb/ft$^3$)</th>
<th>$\phi$ Friction</th>
<th>Cohesion (psf)</th>
<th>$\psi$ Dilatation</th>
<th>$E$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>130</td>
<td>40°</td>
<td>300</td>
<td>12.5°</td>
<td>1.0E6</td>
</tr>
</tbody>
</table>

The piles and soil below the pile cap were not considered in the model to concentrate deformations and forces in the pile cap and above. A rigid-elastic incompressible solid with dimensions of the
superstructure end diaphragm was used to apply the monotonic displacement in the longitudinal and transverse directions (independently). The rigid body was tied to the bearings directly to transfer load via the stem wall. The elastomeric bearing pad dimensions were 0.45×0.45×0.065 m. Bearings shear and elastic modulus equal to 1.034 MPa and 34.47 MPa, respectively. The yield and ultimate strain equal to 150% and 300% of the bearing thickness. Poisson’s ratio equal to 0.475 was used of the bearings material. A friction interaction after gap closure was defined between the rigid body and back wall and shear keys. A concrete-concrete friction coefficient was assumed in the latter friction interaction. Back wall-superstructure and shear key-superstructure gaps, according to MGR bridge drawing, that equal to 0.07 m and 0.027 m, respectively, were defined in the FEM.

Several different modeling configurations were adopted in the continuum models, as shown in Table (4.2). The model identifiers all contain the minimum abutment concrete (AC) structural part of the model. Five modeling configurations were used in the longitudinal direction and four in the transverse direction. In the longitudinal direction, a single node at each pile location was constrained in the longitudinal direction to represent the pile reactions, and all pile cap bottom surface nodes was modeled as roller supports. The abutment transverse resistance was subdivided into three levels. The lower-bound resistance represents the back fill, embankment, and wing walls contribution. The upper-bound resistance represents the resistance in the back wall, stem wall, and pile foundation. Finally, an intermediate level was introduced with springs at the right side of the pile cap with 7 kN/mm/pile stiffness (according to Caltrans [20]) to represent the piles.

Current provisions recommend providing CJs between the stem wall and shear keys and between the shear keys and back wall to ensure the shear keys remain sacrificial. Sufficient reinforcement (between shear key and stem wall) is necessary to ensure the shear keys break off and prevent excessive superstructure displacement and unseating in service-level cases. The reinforcement between the shear key and the stem wall was calculated according to Caltrans [20] for the ACST_SKJ
case.

Table 4.2: Abutment Studied Cases

<table>
<thead>
<tr>
<th>Case</th>
<th>Direction</th>
<th>Soil</th>
<th>Pile cap</th>
<th>Piles</th>
<th>Back wall</th>
<th>Shear key</th>
<th>Wing wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACL</td>
<td>Longitudinal</td>
<td>N</td>
<td>Roller</td>
<td>Pin</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>ACSL</td>
<td>Longitudinal</td>
<td>Y</td>
<td>Roller</td>
<td>Pin</td>
<td>✓</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>ACSL_BJ</td>
<td>Longitudinal</td>
<td>Y</td>
<td>Roller</td>
<td>Pin</td>
<td>✓</td>
<td>✓</td>
<td>None</td>
</tr>
<tr>
<td>ACSL_BKJ</td>
<td>Longitudinal</td>
<td>Y</td>
<td>Roller</td>
<td>Pin</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>ACSL_BKWJ</td>
<td>Longitudinal</td>
<td>Y</td>
<td>Roller</td>
<td>Pin</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>ACST_R</td>
<td>Transverse</td>
<td>Y</td>
<td>Roller</td>
<td>Roller</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>ACST_P</td>
<td>Transverse</td>
<td>Y</td>
<td>Roller</td>
<td>Pin</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>ACST_S</td>
<td>Transverse</td>
<td>Y</td>
<td>Roller</td>
<td>Spring</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>ACST_SKJ</td>
<td>Transverse</td>
<td>Y</td>
<td>Roller</td>
<td>Spring</td>
<td>✓</td>
<td>None</td>
<td>None</td>
</tr>
</tbody>
</table>


4.2.2 OpenSees Finite Elements Model

In the simplified model, the spring abutment model [11] was used. The bearing properties adopted in the FEM were also used in the simplified model. The spring abutment model contains two sets of zero-length elements. The first set were used to represent the longitudinal, transverse, and vertical response of the abutments components that are: expansion joint gap, shear keys, and bearing pads. The second set were used to represent the back fill, back wall, and piles group response in the longitudinal, transverse, and vertical direction. Further information about the abutment components
strength, stiffness, material constitutive model, and elements configuration can be found in Aviram et al. [11]. The hyperbolic force-displacement (HFD) method developed by Shamsabadi [126] was also used to calculate the back fill response and compared with the results for verification purposes. The experimental soil parameters adopted by Stewart et al. [138] were used in the HFD method calculations.

Figure 4.4: OpenSees model for the abutment subsystem

4.3 Abutment Sub-system Results

4.3.1 Longitudinal Direction Results

4.3.1.1 Longitudinal Direction Results

Figure (4.5) illustrates the abutment force-displacement relationship for the adopted modeling configurations. The results of both the continuum and simplified models are presented for each scenario, along with the back wall flexural and shear capacity and the passive soil and structural part
contributions to the total resistance. The simplified model resistance includes piles, back fill, bearing pads, and gap contributions, and does not vary based on the BCs. The general behavior can be characterized by bearing displacement until gap closure engages the back wall or shear keys. Figure (4.6) illustrates the concrete cracking pattern characterized by concrete tension damage at 0.25 m of longitudinal displacement.

Besides the considerable contact area between the abutment and back fill, in most abutment configurations, there is an effective connection between the back wall and other components that results in cantilever response for the abutment. As a result, soil engagement approximately coincides with back wall engagement. A continuous plateau at large displacements is due to residual resistance of the steel in the stem wall and the inability of the Mohr-Coulomb material model to capture the post-peak degradation (softening due to soil dilatancy and de-bonding). The nominal shear and flexural back wall capacities were determined assuming one-way behavior of the back wall with fixed base at the stem wall and free at the top. The total capacity was calculated based on the unit strip capacity and the effective back wall width.
Figure 4.5: Abutment Resistance (longitudinal direction FEM).
Figure 4.6: Abutment subsystem damage at 0.25 m displacement (longitudinal direction FEM)
In this section, the effect of abutment configuration, back wall-components CJs, and components’ contribution to the resistance in the longitudinal direction will be introduced. The significant contribution of the friction components and its effect on the abutment resistance will be discussed subsequently. The ACL results (Figure 4.5(a)) show that the structural resistance is 130% of the simplified model predicted value for the monolithic abutment case. The hand calculated back wall capacities (shear and flexural) alone are equal to the simplified model resistance. The additional back wall capacity shifts the inelastic behavior into the stem wall, as shown by the damage in Figure (4.6(a)). Soil resistance adds approximately 40% above the structural contribution, as shown for the ACLS case in Figure (4.5(b)). The figure confirms the soil is not the only resistance mechanism and the structural components both resist load and accumulate back wall, stem wall, and wing wall damage, as shown in Figure (4.6(b)). The soil resistance contribution obtained from the FEM is consistent with the estimates of the theoretical hyperbolic force-displacement (HFD) method [126].

While introducing a back wall-stem wall CJ in case ACSL_BJ (Figure 4.5(c)) does not affect the soil resistance, it reduces the structural portion and total abutment resistances by 35% and 50%, respectively. The reduction in the force transferred to the stem wall reduces the damage, but engages the wing wall more (Figure 4.6(c)). It should be noted that the results of the HFD and simplified models shown in Figure (4.5(b)) coincide with ACSL instead of ACSL_BKWJ. Such behavior can be attributed to the fact that both (Caltrans and HFD) were calibrated based on the same experimental tests [117] on end-diaphragm abutments (did not include CJ). The tested abutment consisted of piles and monolithic back wall, stem wall, and wing walls with sufficient reinforcement between components. Despite some differences in abutments’ configurations, such low resistance of ACSL_BKWJ configuration was also observed by Bozorgzadeh [16].

Providing shear key-back wall CJs in case ACSL_BKJ reduces the soil and structural contributions to the total resistance (Figure 4.5(d)). However, the reduction is less than 10% due to the minor
effect of this interface area on engaging the stem and wing walls in the response, and leads to a slight reduction in the damage (Figure 4.6(d)). Finally, completely separating the back wall from the other components in ACSL_BKWJ substantially reduces the structural damage and resistance, Figures (4.5(e) and 4.6(e)), and shows a possibility of back wall failure (sacrificial behavior) as recommended by design codes [20]. However, there is still about 10% underestimation in the resistance, and the structural portion represents more than 55% of the total abutment resistance.

Figure (4.7) illustrates the resistance contribution of the interface between the structural components (e.g. back wall CJ) and reinforcement passing these areas comparing to the total resistance of the structural portion only. The percentages in the figure were obtained by comparing the reaction forces in the pile cap base (total structural resistance) for the longitudinal modeling configurations. Table (4.3) illustrates the soil and aforementioned resistances comparing to the total abutment resistance (soil and structural portions).
Figure 4.7: Contribution of the abutment’s structural components with respect to the total structural resistance (longitudinal direction)

Table 4.3: Contribution of the abutment components to the total resistance (longitudinal direction)

<table>
<thead>
<tr>
<th>Contribution</th>
<th>ACSL</th>
<th>ACSL_BJ</th>
<th>ACSL_BKJ</th>
<th>ACSL_BKWI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>40%</td>
<td>54%</td>
<td>53%</td>
<td>44%</td>
</tr>
<tr>
<td>C₁</td>
<td>20%</td>
<td>16%</td>
<td>16%</td>
<td>19%</td>
</tr>
<tr>
<td>C₂</td>
<td>30%</td>
<td>23%</td>
<td>23.5%</td>
<td>28%</td>
</tr>
<tr>
<td>C₃</td>
<td>4%</td>
<td>3%</td>
<td>3%</td>
<td>5%</td>
</tr>
<tr>
<td>C₄</td>
<td>5.4%</td>
<td>4%</td>
<td>4%</td>
<td>4%</td>
</tr>
</tbody>
</table>

Friction between the soil and structural portions of the abutment has a considerable effect on the results. The sacrificial back wall assumption considers the back wall translation only and neglects possible tilting and rotation. Due to the tilting and rotation that were observed in the abutment behavior, the vertical components of the soil normal and tangential resistance had a considerable
contribution to the resistance. The FEM results showed that the stem and back walls deform like a cantilever; therefore, the vertical component of the back fill-abutment friction does not enter the simplified model predictions as it should. Moreover, the friction between the wing walls and stem wall and soil is also not commonly considered. Figure (4.8) shows an illustration of the frictional resistance components and force-displacement curves of the friction components relative to the total abutment resistance. The contribution of the friction terms in ACSL, ACSL_BJ, and ACSL_BKJ can reach up to 30% of the total subsystem strength while it is more than 10% for ACSL_BKWJ. Such outcome reveals that the back wall does not move horizontally even in the back wall sacrificial response and the friction vertical component effect even in case of back wall failure. It is important to notice the contribution of the resultant of the vertical components, which result due to abutment tilting and are commonly neglected due to the sacrificial back wall assumption, that represent about 50% of the soil resistance and 25% of the total resistance.
Figure 4.8: Frictional resistance components (longitudinal direction cases)
Figure 4.9: Abutment Global System Response

Figure (4.9) shows the resistance of the abutment subsystem for all the FEM configurations and OpenSees model. The figure reveals the fact that adopting the modeling simplifications underestimate the actual resistance and that could reach up to 60% in some cases. The figure also unveils several facts such as all the abutment’s components are contributing to the resistance, the back wall is not always a sacrificial component, and back fill does not contribute significantly to the resistance and not the only resisting element to the superstructure displacement. Besides, it is important to notice that, due to the use of pinned BC in the piles locations, the forces is concentrated in the abutment components. Considering the piles in the model, though, may reveal other effects such as piles’ undesirable response.

The additional system resistance, that was observed in the results, could be useful because it would reduce the superstructure longitudinal movement in seismic events. Though, it could also induce considerable shear forces in the abutment that would cause undesirable consequences such as stem wall damage as shown in the abutment’s damage levels illustrated in figure (4.6) that
shows the concrete damage due to tension in the abutment subsystems. Moreover, such additional forces could be mobilized to other critical components such as the piles leading to substantial effect on these components such as shifting their behavior from the elastically assumed response to unconsidered inelastic behavior.

It is important to remember here that the piles’ locations were modeled as fixed support (should not have any lateral displacement) and this does not reflect the actual behavior. In the actual behavior, as illustrated above, the additional system forces would cause considerable consequences such as excessive piles’ displacement or ground soil failure as observed in several real events [63, 67, 122, 145].

4.3.1.2 Back Wall Behavior

As design codes’ specifications assume back walls as sacrificial elements with a failure mode that characterized by shearing off and longitudinal movement into the back fill, studies showed that back walls are not sacrificial with considerable effect on the other components and abutment’s overall behavior (Luo et al. [73]). Figures (4.10 and 4.11) show the back wall deformation obtained from the abutment’s FEM in the longitudinal direction. It can be observed that in case ACSL a cantilever action of the abutment was obtained which can be attributed to the effect of the connection between the back wall and the other components (shear keys, stem wall, and wing walls). Introducing a back wall CJ (ACSL_BJ) reduces that effect with minor separation between the back wall and stem wall, though, there is still a noticeable longitudinal deformation in the abutment. In the third case, shear keys-back wall-stem wall CJ (ACSL_BKJ), a noticeable disconnection of the back wall was observed with a moderate effect on the abutment behavior. Moreover, the deformed shape shown in figure (4.11) clearly illustrate the downward movement of the failure plane with increasing the interface regions of the back wall and the other abutment’s components. It is also
obvious that the deformation distribution is moving upward and concentrating in the region behind the back wall for the shear key-back wall-stem wall CJ. Such behavior also confirm the significant effect of the back wall on the response. Despite the low probability of back wall failure in ACSL_BKJ, the cases above confirm that the back wall is not breaking especially with the adopted design configurations. Such configurations that include back wall-wing wall monolithic connection, back wall-wing wall reinforcement, and significant back wall-stem wall reinforcement have a substantial effect on the back wall failure mode that altering the behavior and causing a unpredictable response. Accordingly, special design configurations should be specified for the back wall connections with other components to ensure the assumed failure mode. Using back wall-wing wall CJ (ACSL_BKWJ) completely separating the back wall and concentrating the back fill deformations behind the back wall. ACSL_BKWJ also show a good possibility of back wall sacrificial behavior as recommended by design codes, yet, there is still additional resistance in the subsystem and such configuration is not commonly adopted in bridges. Such additional resistance is due to the considerable amount of reinforcement between the back wall and wing walls. Accordingly, further assessment also required for the revisions related to back wall-wing walls reinforcement.
Figure 4.10: Back Wall Deformation for the Abutment Longitudinal Cases
Figure 4.11: Abutment Subsystem Deformation Shape
An important indication that helps to understand the actual back fill response, failure mechanism, and failure plane is the soil shear strain. Figures (4.12 to 4.14) illustrate the contours of the soil incremental shear strain ($\gamma_{xy}$) at three different displacement levels for the abutment’s longitudinal cases. The figures also show the growth of the shear strain with increasing the displacement that provides a good indication for the possible failure surface of the back fill. It is obvious from figure (4.12) the considerable effect of the back wall-stem wall monolithic connection and the development of the strain beyond the stem wall that introduces alternative resistance resources, failure mechanisms, and load paths which in turn significantly alter the back fill response. It can be seen in figures (4.13 through 4.15) that a specific failure with a log-straight shape is the resulted failure shape that dominates throughout the loading procedure. The latter figures also show a concentration of the strains in the back wall-stem wall interface and the strain development through the back fill that provides a very good estimation of the back fill failure mode (failure plane and mechanism). Yet, it is important to remember that, even in the abutment CJ cases, there is strain development in the back fill region beyond the stem wall that unveils alternative possible response. Such response include the stem wall engagement and its corresponding effect on the whole abutment response. The reasons for such different response are the back wall-wing wall interface, wing wall-stem wall interface, and back wall-stem wall reinforcement besides the reinforcement in the latter interface regions.
Figure 4.12: Incremental Shear Stain ($\gamma_{xy}$) of Abutment (ACSL)
Figure 4.13: Incremental Shear Stain ($\gamma_{xy}$) of Abutment (ACSL_BJ)
Figure 4.14: Incremental Shear Stain ($\gamma_{xy}$) of Abutment (ACSL_BKJ)

(a) ACSL_BKJ (1.2 inch Displacement)

(b) ACSL_BKJ (2.4 inch Displacement)

(c) ACSL_BKJ (4.0 inch Displacement)
(a) ACSL_BKWJ (1.2 inch Displacement)

(b) ACSL_BKWJ (2.4 inch Displacement)

(c) ACSL_BKWJ (4.0 inch Displacement)

Figure 4.15: Incremental Shear Stain ($\gamma_{xy}$) of Abutment (ACSL_BKWJ)
4.3.1.4  Abutment Subsystem Results Verification

Bozorgzadeh [16] tested a seat-type abutment, with back wall that is sheared off from the stem wall and wing walls, under cyclic load. The test results were compared with two experimental tests that were tested by Maroney [81] and Rollins and Cole [114]. In addition, the test results were also compared with the hyperbolic model developed by Duncan and Mokwa [33]. Maroney [81] carried out a cyclic test on two large scale diaphragm abutments. The west (first) abutment was tested in the longitudinal direction while the east (second) abutment was tested in the transverse direction. Rollins and Cole [114] conducted a full scale static load tests on a concrete pile cap with back fill. Four soil types were used in Rollin’s tests that are sand, silty sand, fine gravel, and coarse gravel. Figure (4.16) shows a comparison in the force-displacement curves between Caltrans [21] method, Duncan and Mokwa [33] model, a model developed by Bozorgzadeh [16], and the experimental tests conducted by Maroney [81] and Rollins and Cole [114], respectively. It can be noticed that Caltrans method, that is based on Marony’s tests, underestimate the abutment stiffness and strength of the back fill. Such comparison verify the obtained results in the abutment subsystem that clearly reveal such underestimation. Figure (4.4) shows the incremental shear strain contours obtained in Bozorgzadeh [16] tests. It can seen that the shear stain distribution and failure surface prediction obtained in ABAQUS FEM agrees with the results obtained from the finite elements analyses performed by Bozorgzadeh [16]. Such agreement also verify the abutment subsystem results and confirm the models ability to capture the actual response.
Figure 4.16: Measured and Calculated Force-Displacement Curves (Bozorgzadeh [16])

(a) The Comparison with Maroney’s test  
(b) The Comparison with Cole and Rollin’s test

Figure 4.17: Incremental Shear Strain Contours

(a) Shear Strain Contours (Test 1 Bozorgzadeh [16])  
(b) Shear Strain Contours $\gamma_{xy}$ (ABAQUS Model)
4.3.2 Transverse Direction Results

Figure (4.18) illustrates the abutment force-displacement relationships for the adopted modeling configurations. The results of both the continuum and simplified models are presented for each scenario, along with the passive soil and structural contributions to the total resistance. The simplified model resistance here includes back fill, wing walls, bearing pads, and shear key resistance, and does not vary based on the BCs. The general behavior can be characterized by superstructure movement on the bearings until gap closure (if gap is present) followed by a dual load path through the shear key and bearings. The forces transferred through both the bearings and shear key are resisted by a combination of the stem wall, back wall, and wing walls/soil. Figure (4.19) illustrates the concrete cracking pattern characterized by concrete tension damage at 0.25 m of transverse displacement.
Figure 4.18: Abutment resistance (transverse direction FEM)

Case ACST_R includes the resistance of the soil through the wing wall only and caused damage in the stem wall-wing wall-back wall connection area, as shown in figures (4.18(a) and 4.19(a)). The wing wall and soil resistance in ACST_R represent 50% of the simplified model resistance. Such outcome confirms the substantial contribution of the wing wall in the transverse resistance and supports the consideration of the wing wall effectiveness factor in estimating the simplified response.
Using pinned BC in ACST\_P (Figure 4.18(b)) increased the resistance by 70% over the simplified resistance due to contribution of the structural components. The attraction of forces to the stem wall altered the failure mode causing severe damage in the stem wall and back wall as shown in Figure (4.19(b)). The wing wall and soil resistance is never mobilized due to the pinned connections (no movement). Similar behavior occurred in ACST\_S, which includes the pile stiffness into the subsystem stiffness and strength. Due to the engagement of all components in the resistance, considerable damage is induced in the entire subsystem as shown in figure (4.19(c)). The flexibility of the piles does not alter the strength or the failure mode, just delays the displacement at the onset of damage. The unchanged strength between ACST\_P and ACST\_S can be attributed to the trade-off in contributions between the structural components and soil due to the pile flexibility.

The three BC cases all exhibit substantial damage at 0.25 m displacement (Figure 4.19), particularly the stem and back walls and wing wall-abutment interface. No significant reinforcement yielding or concrete damage occurs in the shear key connections. Such behavior could be also attributed to the excessive shear key resistance resulting from the integrated shear key configuration and significant amount of reinforcement between the shear keys and both stem wall and back wall. Naturally, the introduction of the CJs in ACST\_SKJ (figure 4.19(d)) substantially impact the resistance and damage and limits the subsystem resistance to that of the shear key-stem wall interface reinforcement. Such a configuration protects the other abutment components, yet, the existing reinforcement still engages the stem wall. The result is a 40% increase in resistance in the subsystem as shown on figure (4.18(d)).
Comparing the abutment structural resistance (reactions in the abutment base) shown in the previous results reveals that the structural components provide 75% of the actual subsystem resistance transferring it to the foundation comparing to less than 25% for the soil. Figure (4.20) illustrates the resistance contribution of the interface between the structural components and reinforcement passing these areas comparing to the total resistance of the structural portion only. Table (4.4) illustrates the soil and aforementioned resistances comparing to the total abutment resistance (soil and structural portions). It should be mentioned that in ACST_R, ACST_S and ACST_SKJ, due to the major structural components effect and stem wall-wing wall-back wall connection damage, reasonable strains were observed in the back fill soil especially after wing wall damage. Negligible
soil strains were observed in ACST_P.

![Diagram of abutment structural components](image)

**Figure 4.20:** Contribution of the abutment structural components with respect to the total structural resistance (transverse direction)

<table>
<thead>
<tr>
<th>Contribution</th>
<th>ACST_R</th>
<th>ACST_P</th>
<th>ACST_S</th>
<th>ACST_SKJ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>87.5%</td>
<td>0%</td>
<td>17%</td>
<td>25%</td>
</tr>
<tr>
<td>C&lt;sub&gt;1&lt;/sub&gt;</td>
<td>9.5%</td>
<td>75%</td>
<td>62%</td>
<td>18.75%</td>
</tr>
<tr>
<td>C&lt;sub&gt;2&lt;/sub&gt;</td>
<td>3%</td>
<td>25%</td>
<td>38%</td>
<td>6%</td>
</tr>
</tbody>
</table>

**Table 4.4:** Contribution of the abutment components to the total resistance (transverse direction)

Finally, figure (4.21) shows a comparison between the abutment FEM with construction joint, simplified model, and the abutment FEM with the adopted boundary condition configurations. It can be noted from the figure that even with the existence of the construction joint, there is still additional resistance in the system that could be attributed to the stem wall contribution which can also be observed in the abutment damage shown in figure (4.19(d)). This contribution could be...
attributed to the shear key-stem wall reinforcement that mobilize the force to the stem wall and
does not provide the proposed failure mode.

![Graph](attachment:image.png)

Figure 4.21: Abutment Subsystems Results (Transverse Direction)

4.4 Columns-Bent Subsystem

A bent is an intermediate support between bridge spans that supports the superstructure girders
and transfers the vertical and lateral loads from the superstructure to the foundation. Bents may
be categorized into a single-column and multi-column bents. Multi-column bents consist of a cap
beam, bent columns, and supporting foundation such as pile cap and piles. The cap beam is the
element that connect the bent columns with each others and transfer the loads from the longitudinal
girders to the bent columns.

Seismic design codes consider column demands, such as displacements and moments, as the con-
trolling limit states for bridges’. As a matter of fact, column bent failure and seismic response
were considered as the main defense line against superstructure and other critical components fail-
ure. Such objective can be achieved by concentrating the main inelastic behavior in columns and maintaining elastic response for the other components. It was found by many studies that using the one-dimensional elements in bent cap modeling significantly reduces its torsional rigidity and increases its flexural flexibility. McCallen and Romstad [84] concluded that using the beam elements model fail to capture the bent cap rigidity and caused excessive flexibility and deformation in the cap beam. The authors increased the bending section properties by three order of magnitude in order to prevent excessive deflection of the center cap beam. This impact on the flexural and torsional stiffness on bridges’ response is more pronounced and obvious in bent caps with two columns or more.

The bent cap in La Veta Avenue Overcrossing Bridge was selected to study the effect of using the one-dimensional elements in modeling bent caps. La Veta Avenue Overcrossing is a two-span bridge with a total length of 91.4 m that is divided into 47.2m and 44.2m individual spans. The bridge consists of six cells continues concrete box girder superstructure, seat-type abutments, a column-bent with two reinforced concrete columns and integrated cap beam, and pile foundation system that consists of pile cap and 20 CIDH piles (Kaviani et al. [62]). The other properties of the aforementioned bridge are illustrated in table (4.5). La Veta Avenue Overcrossing column-bent was modeled in both ABAQUS and OpenSees. A full bridge with full superstructure length was modeled as a FEM and simplified 3D spine model to insure the inclusion of the superstructure modeling effect on the behavior and avoid the consequences of using segments of the superstructure on the response. The superstructure boundary conditions was modeled as roller support to exclude the effect of the boundary conditions on the responses.
Table 4.5: La Veta Avenue Overcrossing properties

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total deck width</td>
<td>23.0 m</td>
</tr>
<tr>
<td>Deck depth</td>
<td>1.9 m</td>
</tr>
<tr>
<td>Columns clear height</td>
<td>6.7 m</td>
</tr>
<tr>
<td>Columns Diameter</td>
<td>1.7 m</td>
</tr>
<tr>
<td>Deck Centroid</td>
<td>1.04 m</td>
</tr>
<tr>
<td>Length of cap-beam to column’s centroid</td>
<td>5.5 m</td>
</tr>
<tr>
<td>Cap-beam dimensions (B x H)</td>
<td>2.3 m x 1.9 m</td>
</tr>
<tr>
<td>Superstructure concrete material properties (E_c, f'c)</td>
<td>(27800 MPa, 34.5 MPa)</td>
</tr>
<tr>
<td>Columns material properties (f'c = 34.5) MPa, Steel: ASTM A706</td>
<td></td>
</tr>
<tr>
<td>Column longitudinal reinforcement</td>
<td>44 #36 (bundles of 2)</td>
</tr>
<tr>
<td>Column transverse reinforcement</td>
<td>Spirals, #13 @ 152 mm</td>
</tr>
</tbody>
</table>

4.4.1 ABAQUS Finite Element Model

The full bridge was divided into three segments in the FEM. The first segment that include the columns, cap beam, and a segment of superstructure with a length that correspond to the dead load moment inflection point was modeled using concrete damage plasticity model to include the concrete stiffness degradation due to compression and tension in these components. The rest of the superstructure was modeled using elastic concrete materials. The pile cap was also modeled using elastic concrete material.

The concrete and steel element’s types, elements section controls, and analysis procedures de-
scribed in section (3.6) were used in the FEM. Radial mesh was used for the columns with a confined concrete material for the core and unconfined concrete for the cover. The concrete material constitutive relationships and material models and parameters described in sections (3.4, 3.10.2.1.1, and 3.12.2.1.1) were used in the columns bent study. The reinforcement steel material was modeled following section (3.6) with yield stress equal to 470 MPa. ABAQUS finite element model for the full bridge is shown in figure (4.22). A monotonic displacement control loading procedure was used in the FEM up to a displacement of 0.25m.

4.4.2 OpenSees Finite Element Model

The bridge was modeled in OpenSees using nonlinear beam column elements with nonlinear fiber section for the columns and elastic section for the cap beam and superstructure. One element with ten integration points was used to model each column. The columns cross section were discretized into 22 angular divisions and 24 radial divisions with two radial divisions for the concrete cover. Mander et al. [78] confined concrete model with concrete02 uniaxial material model was used to model the concrete core in the columns while unconfined concrete was used for the columns’ cover. The concrete material parameters and constitutive relationship adopted for the column and T-girder components’ calibration were used in the simplified model of the bridge, with modification that correspond to the material properties assumed in La Veta Over crossing design process.

The superstructure were modeled using two elastic beam column elements with 5 integration points in each element. It is important to mention that the full superstructure length were modeled using elastic section while a nonlinear segment of the superstructure was used in the FEM. Such simplified model configuration was used to achieve the recommended assumptions by design codes that are commonly adopted in bridges’ design and assessment procedures. The superstructure elements were defined at the elevation of the superstructure centroid and a separate segment was defined
at the column top to represent the column portion embedded in the cap beam. This segment was defined as a rigid offset at the column with a rigidity factor equal to 1 to account for the high stiffness provided by the joint between the column and the deck. The segment length was equal to the distance between the column height elevation and the superstructure centroid level. Big value was assigned for the superstructure torsional stiffness due it’s negligible effect on the subsystem behavior.

The cap beam was modeled using four elastic beam column elements (five integration points each) with a rectangular solid section with the dimensions equal to 2.3 and 1.9 for the width and height, respectively. Two elements were used for the cap beam central part and one element was used for each cap beam wing. Also, since the superstructure and cap beam are usually built monolithically, a rigid connection was used to model the connection between the superstructure and cap beam in order to represent the actual connection behavior. OpenSees model for the bent cap subsystem is shown in figure (4.22). The boundary conditions represented by using a roller support in both superstructure ends and fixing the columns bottom nodes. The column subsystem loads included the gravity load followed by a displacement controlled loading procedure at both superstructure ends.
Figure 4.22: Column Bent Models

(a) ABAQUS Model

(b) OpenSees Model
4.4.3 Bent Cap Sub-system Results

As mentioned, the two modeling approaches were developed to determine the effect of using of one-dimensional spine elements in cap beam and superstructure modeling. Figure (4.23) show the superstructure curvature for the FEM and simplified model approaches. It can be noticed that the simplified model results are close to the FEM results. Though, due to the use one-dimensional spine element there is a difference in the superstructure deformations about 10% to 60% in some points. Such behavior would induce reduction in the other components deformations causing underestimation for the system responses. Figure (4.24) shows the columns curvature for both modeling approaches. It is obvious that the curvature in the simplified model is significantly higher than curvature in the finite elements modes. This behavior can be attributed to the use of the one-dimensional element that induces excessive flexibility not only in the components but also in the total system behavior.
The effect of the simplified models on the cap beam behavior is shown in figure (4.25) that illustrates the cap beam twist, deflection, and rotation for both modeling approaches. Several modeling simplification consequences can be observed in the figure. First, the cap beam twist and deflection in the simplified model is higher than the FEM. Second, the cap beam twist in the finite element model is negligible while an excessive twist was observed in the simplified models that is more than 400% of the twist in the finite element models.

Figure 4.24: Column Curvature for full bridge and subsystem cases
By comparing the cap beam deflection and rotation for the two models, it can be noticed that using 1D element in the cap beam modeling induce excessive flexibility indicating unrealistic cap beam failure. These failure mode include torsional and flexural failure, though, neither failure mode is the actual behavior. This behavior could be attributed to the infinitesimal connection between the superstructure and cap beam that concentrate the damage at a single point instead of distributing it on the whole member length. The unrealistic representation of the cap beam that reflect its effective width and torsional rigidity is another source for the simplified model deficiency.
It is important to point out here the behavior differences between the simplified models and the finite elements model. As observed, there is an excessive cap beam deformations (twist, rotation, and deflection) in the simplified models. On the contrary, the superstructure and cap beam have
negligible deformations in the FEM while the columns undergo most of the considerable deformations. Figure (4.26) shows the tension damage and stiffness degradation in the finite elements models. The figure clearly shows that the cap beam and superstructure segment are not subjected to any kind of damage comparing to an obvious sever damage in the columns. Figure (4.27) shows the longitudinal displacement of the cap beam section in the finite elements models. The figure also confirms the previous results which are the negligible rotation of the cap beam and superstructure and the concentration of the effect on the columns.

(a) Tension Damage

(b) Stiffness Degradation

Figure 4.26: Columns Bent Damage Variables
Figure 4.27: Cap Beam Horizontal Displacement for the Finite Elements Model

4.4.3.1 Column Subsystem Results Verification

In order to obtain further confidence in the column bent subsystem results, a simple comparison was performed between the FEM results of the columns bent FEM results, experimental results of the column-bent cap tests conducted by Moustafa and Mosalam [93], and finite element analyses results of the mentioned experimental tests that were performed by the same author. The FEM were created using the finite element package DIANA Diana [31]. It was observed in ABAQUS FEM that most of the damage concentrates in the columns that experience a flexural plastic hinges at both ends without noticeable damage in the superstructure and cap beam. The concrete damage measure (degradation in the elastic stiffness) obtained from ABAQUS model indicates concrete cracking due to tension or compression, spalling, and stiffness degradation of the concrete elements. The
observed mode of failure, the cap beam and superstructure remain essentially elastic and plastic hinge formation in the columns, satisfies Caltrans SDC design objectives. This mode of failure and observation was also noticed in the experimental test results in which a plastic hinge failure was formed in the column top end, that also experiences cracking and spalling of the concrete, with minor cracks in the superstructure top surface due to the high self weight percent adopted during the cyclic load. Furthermore, the finite element models of the test specimen predicted the same failure mode and concrete damage in the tension side of the column that predicted concrete cracking, spalling, and stiffness degradation in the column top [93]. Figure (4.28) illustrates the columns damage observed in ABAQUS FEM, experimental test results, and FEM results of the test specimen. Due to the significant difference between the experimental test and FEM developed in ABAQUS (column, cap beam, and superstructure shape and dimensions), further comparison could be obtained between the FEM and experimental results.
4.4.3.2 Effect of the Cap Beam Parameters

As noticed, and according to the cap beam immoderate responses and incorrect predicted behavior, the primary deficiency is the cap beam properties and modeling assumptions that are adopted in the simplified models. As a result, two cap beam parameters were investigated to determine the effect of the cap beam assumed parameters on the behavior. These parameters are the cap beam
torsional and flexural rigidity. Three cap beam torsional and two flexural stiffnesses were used in the simplified models. The considered torsional stiffnesses were high torsional stiffness value, the actual cap beam stiffness, and 25% of the actual cap beam stiffness as recommended by the ATC [9]. For the flexural stiffness study, the actual cap beam stiffness was compared with an increased stiffness (by three orders of magnitude) that was used to eliminate its excessive flexibility [84].

Figure (4.29) illustrates the cap beam deformations for the considered torsional stiffness parameters for both modeling approaches. It can be observed from the figure the considerable twist, deflection, and rotation that result due to the use of the actual cap beam and design codes recommended stiffnesses. It can be also noticed that increasing the cap beam torsional stiffness reduces the twist for both cases. However, it will still induce excessive flexibility and deflection increase for the bridge system. This behavior, comparing to the FEA results, reveals that none of the considered cap beam stiffnesses provide an accurate prediction for the actual system response that observed in the finite elements models.
The effect of the cap beam flexural stiffness on the cap beam behavior is shown in figure (4.30) that illustrates the cap beam deformations for both models and two stiffness values (actual stiffness and increased stiffness). It can be observed from the figure that increasing the flexural stiffness reduces the cap beam deflection effectively. Still, there is an excessive cap beam twist in the simplified
Figure 4.30: Cap Beam Deformations for the full bridge case

Figure (4.31) shows the effect of changing the cap beam parameters, comparing to the FEM results, on the columns deformations. It can be noticed that the flexural stiffness has no effect on the
columns behavior in the longitudinal direction. However, it can be observed that the torsional stiffness has a considerable effect on the columns deformations. Adopting the design code recommended value [9] underestimate the deformations while adopting a high value for the torsional stiffness overestimate the columns deformations. Yet, both of the obtained responses do not reflect the actual system response.

![Torsional and Flexural Stiffness Effects](image)

**Figure 4.31: Column Deformations for the Cap Beam Parameters Study**

The columns bent results reveal that the spine models shortcomings are not only due to the assumed components’ properties but primarily because of the modeling assumptions and simplifications.
These deficiency resources can be characterized by the use of one-dimensional spine elements that induce flexibility and the infinitesimal connection between the superstructure and cap beam elements that concentrate the deformations causing inaccurate prediction for the response. Moreover, the bent cap results unveil the considerable overestimation of the bridge system responses due to the use of the spine models and the substantial gap in the predicted response that could result due to such responses. For instance, using such cap beam deformations in a design or assessment process, depending on the engineering judgment, may be considered as a cap beam failure that would be prevented by an increase in the cap beam section capacity which would have unconsidered drawback effect on other components such as the columns. The underestimated/overestimated system stiffness and strength, as another example, could lead to inaccurate quantification for the system capacity and demand which in turn would increase the possibility of deficient design or assessment process.

4.5 Chapter Discussion

As seen in the abutment subsystem results, comparing to the FEM results, the use of modeling simplifications and load path and failure mode assumptions recommended by design codes induce significant underestimation to the actual subsystem resistance. Such underestimation can reach up to 60% and 70% in the longitudinal and transverse direction, respectively. Such consequence could be attributed to the neglect of the substantial contribution of the unconsidered abutment’s components, the structural components, in particular. Besides, despite that such additional force could have a positive outcome such as limiting the longitudinal superstructure movement in seismic events, it was observed that such excessive force could shift the failure mode inducing considerable damage in abutments’ critical components. Moreover, such additional forces could be mobilized to other critical components such as the foundations leading to a substantial effect on
these components such as shifting their behavior from the elastic to inelastic behavior.

These results shed light on some of the potential problems that could result due to such unpredicted excessive strength. For instance, such high resistance and due to the impact between the superstructure and abutment, could result in unpredicted local or global failure in the deck, diaphragms, and pre-stressed girders. Also, since the abutment will continue to provide resistance in the longitudinal direction reducing the displacement in this direction, the displacement in the transverse direction will be much larger than the longitudinal displacement. Such difference, in turn, will consternate the damage in the transverse direction leading to several unpredicted problems least of them are columns failure and/or superstructure unseating as observed in seismic events [67]. Finally, the unconsidered resistance could increase the amount of forces mobilized to the foundations leading to unpredicted damage in the piles and/or ground soil (not considered in the current study) which has been observed in several seismic events [122].

As a design point of view, the modeling simplification that assumes instant shear keys and back wall disconnection and the load transfer to the abutment’s back fill will severely overestimate bridges’ displacement demand. This, in turn, will lead to inaccurate and unrealistic structural responses. Using such results in a design process will highly increase the structural seismic demand and in turn, exaggerate members capacity and demands adopted in the design. The new members capacity will add even more resistance to the system (more underestimated force) and more unrealistic response. Accordingly, aforementioned consequences necessitate further assessment to the load path and failure modes that are used in the simplified models, the design codes hypotheses that are related to the load path and failure modes and recommended in bridges design, and the effect of the other bridge components and their contribution to the local and global response.

For the column bent case, the excessive torsional deformation and flexibility of the cap beam would induce underestimation for the columns forces and deformation that will cause false judgment and
incorrect decisions regarding columns design and assessment. Also, such flexibility in cap beam or superstructure will encourage the designers to enlarge their capacity (bigger sections) which in turn will further increase the gap between the actual and predicted behavior. Finally, adopting high stiffness values for the column bent components would considerably increase the subsystem stiffness and strength that could alter the load path and shift the failure mode to undesirable failure mode. One of the counter effects that could result due to such assumptions is mobilizing the load to the columns’ foundation that would lead to inelastic behavior or unpredicted damage in other critical components such as the piles. Accordingly, it is recommended that special care should be considered in the cap beam modeling approach and its torsional and flexure resistance assumptions due to their substantial effect in shifting the failure modes. Also, it is strongly recommended to adopt alternative modeling approaches for the connection between the components (superstructure and cap beam) to eliminate the consequence of the responses’ concentration in these connections.
CHAPTER 5: EFFECT OF SUBSYSTEMS FINDINGS

5.1 Introduction

As observed in the previous chapters, the design codes hypotheses and simplified models that are used in bridges design and assessment induce significant underestimation of the system resistance that would cause undesirable effect on bridges seismic response. This can also be investigated and confirmed using probabilistic seismic evaluation approaches such as Probabilistic Seismic Demand Analyses (PSDA) and Incremental Dynamic Analyses (IDA). In this chapter, the focus will be on studying the effect of the abutment’s properties (stiffness and strength) in the longitudinal and transverse direction on the behavior of the bridge and the consequences of using the simplified modeling techniques on bridges seismic demand prediction. Two approaches were used to confirm the effect of simplified models disadvantages. First, a set of 1D single-degree-of-freedom systems (SDOF), that have a different configuration in each direction were developed and used in a nonlinear time history analyses to study the effect of the abutment parameters in each direction, separately. In the second approach, 3D bridge models were created using the simplified modeling techniques and used in a nonlinear time history analyses for the same mentioned purpose. After the abutment subsystem study, California’s Murphys Grade Road (MGR) bridge was used as a typical bridge structure to study the effect of the abutments stiffness (i.e., embankment initial stiffness) and strength (i.e., maximum passive pressure) on the bridge dynamic response.

5.1.1 Earthquake Ground Motion Records

The ground motion bin approach was used in the PSDA and the PSDM formulation. This approach was proposed and used by Shome [133] to subdivide the ground motion to bins depending on the
magnitude, distance to fault, and soil type. The advantages of this approach are limiting the number of ground motions selected for the analysis (Shome [133]) and ability to estimate the effect of the generalized earthquake properties (e.g., duration) on the structural demands.

Eight bins with 20 ground motions each were used in the analysis. These ground motion bins were taken from Mackie and Stojadinović [75]. The bins are divided into two groups that are: ground motions recorded on NEHRP soil type D sites and ground motions recorded on NEHRP soil type C sites. They were also categorized into small and large distance ground motions and small and large magnitude ground motions. The threshold of small and large distance and small and large magnitudes were 30 km and 6.5, respectively. Further information about the ground motion processing, categorization, and other ground motion details such as magnitude and fault mechanism can be found in Cronin [28].

In this study, the bridge was oriented to be perpendicular to the fault to use a large number of ground motion records to produce vibration in the bridge response. Also, due to the lack of very high-magnitude records in the utilized ground motion records, and to ensure nonlinear structural response in the bridges, the records were scaled by a factor of two. This was done by simple amplification of the acceleration values during all the Probabilistic Seismic Demand Analyses (PSDA). In the SDOF system, the ground motion records that were applied in the longitudinal and transverse direction of the full bridge model, were used in the analyses. That is, 320 ground motion records in each direction of the SDOF system analyses.

5.2 Single degree of freedom approach

A SDOF system was developed for longitudinal and transverse directions (independently). Each SDOF system consisted of a lumped mass that represents the generalized dynamic mass, and two
parallel zero-length elements to represent the bridge (columns and superstructure) and abutments, respectively. The element representing the bridge was characterized by period and strength, and was defined using an elastic-plastic material (Steel01) with hardening ratio of 0.001. Several periods were assumed to cover a sufficient range of bridge structures: 0.3, 0.5, 1.0, and 1.5 s. The yield strength was obtained from constant ductility analysis. In each iteration, a NTHA was performed by varying the yield strength required until the target ductility value was achieved.

The element representing the abutments had a yield strength assumed equal to 50% of the bridge strength for the abutment stiffness study, and the abutment stiffness equal to 50% of the bridge stiffness for the abutment strength study. The abutment stiffness range was selected to be zero to 150% of the bridge stiffness while the strength range was selected to be zero to 160% of the bridge strength. The element representing the abutment was defined using two elastic-plastic gap materials in parallel (with opposite signs). In the longitudinal direction, the materials simulate the independent compression-only activation of one abutment at a time. The gap was assumed to be equal to zero to avoid the initial gap effect on the system behavior. In the transverse direction, the materials simulate the engagement of both abutments in the same direction simultaneously.

In the longitudinal direction model, a mass equal to one was used. Due to the axial rigidity of the superstructure, the displacements for the bridge and abutment elements were assumed to be the same. Therefore, the ratio used to define the abutment element stiffness was equal to the abutment stiffness divided by the column stiffness. In the transverse direction model, the generalized SDOF approach was adopted to determine the system properties. Due to the flexibility of the superstructure in the transverse direction, a simplified elastic multi-degree of freedom system (MDOF) system with seven DOFs was used. The generalized mass was obtained from the first fundamental (transverse) mode shape. The ratio used to define the abutment element stiffness was obtained assuming a rigid superstructure, i.e., $1^T K 1$, and is therefore comparable to the longitudinal case (a ratio independent of the $EI$ of the superstructure). However, the element stiffnesses were ob-
tained based on the proportion of the generalized first-mode stiffness determined by this ratio, and therefore reflect the appropriate change in the boundary conditions relative to the superstructure flexibility.

5.3 Bridge Dynamic Analyses

In the bridge analyses approach, a PSDM that utilized PSDA approach was applied and used in the bridges analyses. A PSDM is a result of PSDA that relates structural engineering demand parameters (EDP) to seismic hazard Intensity Measures (IM) in a probabilistic framework Mackie and Stojadinović [75]. Such PSDM is practical because the PSDA approach can be applied to a complete class of structures, a group of design or assessment variables, and to an entire seismic region which introduces a quite general prediction model.

The outcome of the PSDA is a cloud of data points that represent individual outcomes of the PSDA. These data points represent the EDP values for a structure subjected to a ground motion with a specific intensity measure IM and by assuming log-normal probability distribution of these outcomes, a linear fit in log space can be used to represent this cloud of points. The parameters of this linear fit define the PSDM, in addition to the dispersion of the data with respect to the linear least-square fit.

5.3.1 Intensity Measure and Engineering Demand Parameter

As mentioned, the PSDM relates the IMs to the EDPs. Engineering demand parameters can be categorize into global such as drift ratio, intermediate such as curvature ductility, and local such as steel stress. Mackie and Stojadinović [75] showed that coupling the drift ratio, maximum column moment, and maximum steel stress with spectral IMs introduce an optimal PSDM because they
are not dependent on the bridge type, the analysis method used in the PSDA, and the variation of
the design parameters. Also, it was shown in FEMA [40] that peak drift angles would have a good
correlation with structural stability, structural damage, and specific types of nonstructural damage.
Accordingly, the drift ratio (i.e., the maximum displacement normalized by column height) and
maximum column moment were considered as the engineering demand parameter of interest in
this research.

Several ground motion IMs can be paired with engineering demand parameters to form a PSDM
such as spectral quantities, duration and energy related quantities, and frequency content charac-
teristics. However, several researchers showed that spectral quantities are very effective ground
motion intensity measures. Shome et al. [134] illustrated that for buildings with period around one
second, $S_{a1}$ represent an effective ground motion intensity measure and predictor for the nonlinear
response. Mackie and Stojadinović [75] confirmed that spectral quantities (i.e., $S_\alpha$) can be superior
IMs due to their direct relation to the model response of the considered structure and their inclusion
to the frequency content measures. As a result of this effectiveness, the peak spectral acceleration
will be considered as the IM of interest in this study.

In each PSDM, the PSDA data points were plotted in log-log scale with the EDP on the abscissa
(x-axis) and the ground motion IM on the ordinate (y-axis). This plot configuration is a standard
procedure in producing a IM-EDP plot and creating a PSDM. By assuming a log-normal distribu-
tion of the EDPs (Shome [133]) the PSDM can be written as in equation (5.1).

$$ EDP = a(IM)^b $$ (5.1)

A linear regression in the log-log space can be applied to equation (5.1) to obtain the PSDM
\[
\ln(EDP) = A + B \ln(IM) \quad (5.2)
\]

Where the coefficient \(A\) can be explained as the value of the EDP (drift ratio) corresponding to a unit of IM (Sa) and \(B\) represent the slope of the PSDM that measure the sensitivity of EDP to the change in the IM and could provide a good estimation for a nonlinear structure softening (i.e., more proportional increase of the drift ratio with the spectral acceleration) and to estimate the effect of the change in the aforementioned parameters over a range of ground motion intensity measures.

In order to utilize the maximum engineering demand parameter for the purpose of this study, the Square Roots of the Sum of Squares (SRSS) method was used to calculate the maximum value of the EDP over the time for each ground motion record. This was also done to consider the combination of the longitudinal and transverse response of the bridge. The SRSS method was also used to calculate the maximum IM using the spectral acceleration at the longitudinal fundamental period \(S_aT_1\) and the spectral acceleration at the transverse fundamental period \(S_aT_2\). Accordingly, each data point, that will be used in forming the PSDM, in the log-log space represent a pair of the max \(EDP_{SRSS}\) and the peak spectral acceleration \(S_a(SRSS)\).

The calculation procedure of obtaining the effect of the aforementioned parameters includes:

1. Performing the nonlinear time history analyses for the SDOF system to determine the maximum EDP-IM pairs (using SRSS method).
2. Forming the PSDM and determining its coefficients using the PSDA cloud of points and linear regression fit in log-log space.
3. In order to obtain the spectral acceleration value for each investigated parameter case (e.g., abutment stiffness), a design spectrum is required to determine the spectral acceleration that corre-
spond to the bridge fundamental period in the longitudinal and transverse direction. Caltrans [20] ARS design spectrum with 7.5 magnitude and PBA equal to 0.7g was used to extract the spectral acceleration for each parameter.

(4) Determining the estimated EDP by substituting the spectral acceleration values obtained in the third step into the PSDM that correspond to each parameter (e.g., abutment stiffness).

5.4 Bridge Model (Abutment Properties Study)

A three-dimensional model of MGR bridge was modeled in OpenSees platform. In addition to determining the abutment’s parameters effect on bridges’ response, this model was also used to capture the response of the total bridge system and each single component during the seismic demand analyses. According to Caltrans [20], the standard initial embankment fill stiffness equal to 14350 N/mm/m and the maximum passive pressure equal to 239kPa. In order to investigate the impact of abutments’ parameters in the longitudinal and transverse direction, a range of the initial embankment fill stiffness and maximum passive pressure that correspond to the parameters range in the SDOF system were used in the full bridge study. The stiffness range is (0-52900 N/mm/m) while the strength range is (0-833 kPa).

5.4.1 Column Model

Same modeling procedure and material models (concrete and steel) described in section (??) was used in modeling the columns in MGR bridge dynamic analyses. A single element with five integration points was used to model each column. The columns cross section were discretized into 12 angular divisions and ten radial divisions with two radial divisions for the concrete cover. The concrete model (Concrete01 and Concrete 04) were used to model the concrete material in the
column cover and core, respectively.

The longitudinal reinforcement included 40-#11 bars that were distributed evenly at the radius of the spiral reinforcement. For the transverse reinforcement, #8 bars were used with 0.15m vertical spacing and 0.05m concrete cover which were used to calculate the confinement factor for the concrete core. The steel initial stiffness equal to 200 GPa with 475 MPa yield strength. The steel ultimate steel strain was taken to be equal to 0.09 for #11 bars and 0.12 for #8 spiral reinforcement (Caltrans [20]).

The column base was modeled as a fixed support while a rigid connection was used to connect the column top with the superstructure. The column mass was calculated using a concrete unit weight of 2286.05 kg/m$^3$ Caltrans [20] and specified to the column top and bottom nodes. In order to consider the column torsional resistance, the column torsional stiffness was included in the model with a cracked section torsional stiffness $J_{eff} = 0.2J$ according to ATC [9].

5.4.2 Superstructure Model

The MGR bridge deck is a prestress reinforced concrete box girder with a two traffic lanes (3.6m width each). It consists of three spans with 33.45 span length for the edge spans and 44.6m span length for the intermediate span. The deck is a three cell box girder with a total width of 12.9m and 1.9m depth. Figure (5.1) shows the bridge deck geometry and dimension. The deck was modeled using elastic section and beam-column elements with area, moment of inertia, and other properties shown in table (5.1). Each span was modeled using two elements with 5 integration points and three masses that were calculate using the deck unit weight ($\gamma=23.56$ KN/m$^3$) and specified to the elements nodes. The rotational mass was also calculated (lumped) and specified to the deck nodes. As a summery, six mass terms were defined for each superstructure node, three translational masses and three rotational masses.
Table 5.1: MGR Elastic Deck Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_g$</td>
<td>$7.083 \text{ m}^2$</td>
</tr>
<tr>
<td>$I_Y$</td>
<td>$3.6939 \text{ m}^4$</td>
</tr>
<tr>
<td>$I_Z$</td>
<td>$86.148 \text{ m}^4$</td>
</tr>
<tr>
<td>$J$</td>
<td>$10.485 \text{ m}^4$</td>
</tr>
<tr>
<td>$f'_c, E_c$</td>
<td>$34.5 \text{ MPa, 27800 MPa}$</td>
</tr>
</tbody>
</table>

Due to elastic behavior assumption of superstructure, the prestress force was neglected in the analyses and a cracked section was assumed for the seismic analyses. Also, as recommended by the ATC [9], the superstructure torsional and flexural stiffness was reduced to $J_{eff} = 0.25J_g$ and $I_{eff} = 0.75I_g$ to account for the cracked section reduced stiffnesses. It should be mention here that the superstructure elements were defined at the elevation of the superstructure centroid and a separate segment was defined at the column top to represent the column portion embedded in the cap beam. This segment was defined as a rigid offset at the column top with a rigidity factor equal to 1 to account for the high stiffness provided by the joint between the column and the deck. The segment length was equal to the distance between the column height elevation and the superstructure centroid level.
5.4.3 Abutment Model

The simplified abutment model proposed by Aviram et al. [11] was used in the abutment model. The bearing properties adopted in the FEM was also used in the simplified model. The spring abutment model contains two sets of zero length elements. The first set were used to represent the longitudinal, transverse, and vertical response of the abutments components that are: expansion joint gap, shear keys, and bearing pads. The second set were used to represent the back fill, back wall, and piles group response in the longitudinal, transverse, and vertical direction. Further information about the abutment components strength, stiffness, material constitutive model, and elements configuration can be found in Aviram et al. [11]. According to Caltrans [20], this should represent all the longitudinal resisting components that are the embankment backfill, bearing pads, backwall, and pile foundation resistance. Figure (5.2) illustrate the abutment model scheme.
5.4.4 Loading and Analyses Procedures

The loading procedure includes two loading cases for each analysis that are gravity load case and dynamic or lateral load case. In the gravity load case, a uniform gravity load was applied at the superstructure and columns elements to represent their self-weight load. The load was applied as a constant one step element load to the specified elements. For the second load case, several bridge configuration were created (to cover the range of the parameters under study) and a group of analyses were performed that include static push over analyses, model analyses, and dynamic time history analyses. Each analysis procedure was performed separately with its corresponding modeling configuration. The analyses procedures include two steps: first the gravity load analysis was performed followed by the second step that includes the other analyses (static or dynamic analyses).

The static analysis procedure includes longitudinal and transverse push over analysis. In this analysis procedure the bearing pads and abutment contribution were neglected and the abutment was
modeled as roller support to determine the yield values for the column and its maximum responses. The analysis procedure was performed by incrementing the lateral load at the columns’ top nodes and measuring the displacement up to column failure. The analysis solution strategy was standard Newton-Raphson strategy.

The modal analysis was performed to determine the mode shapes information and bridge natural frequencies. The first five mode shapes were extracted and compared with the bridge fundamental periods. This was done to insure accurate selection of the controlling fundamental periods for the PSDM. The final analysis procedure is the dynamic time history analysis that usually used to determine the bridge demands values and the maximum dynamic quantities (e.g., displacement, moment, and stress). For each analysis, a static analysis for the gravity loads were performed followed by the dynamic (ground motion) analysis. More information regarding dynamic analyses procedure (e.g. damping model and numerical integrator) can be found in Aviram et al. [11].

5.5 Single Degree of Freedom Results

It was found from the longitudinal NTHA results that, due to the column yielding, increasing the abutment longitudinal stiffness had a negligible effect on the column forces. Figures (5.3(a) and 5.3(b)) illustrate the effect of the abutment stiffness on the ductility, base shear, and abutment strength. The horizontal axis (normalized stiffness) represent the ratio of both abutments’ stiffness to the stiffness of the other lateral resisting components (columns in this case) in the bridge system. It can be seen that increasing the abutment stiffness reduces the system ductility and increases the system forces. The effect of the additional system forces is mobilizing a considerable amount of force to the abutment (changing the load path) and shifting the failure mode from elastic to inelastic abutment behavior as shown in figure (5.3(c)). However, after the abutment and columns yield, further increases in longitudinal stiffness do not substantially change the ductility or total
Sensitivity of responses to the longitudinal abutment strength are shown in figure (5.4) and show similar trends to the variation in abutment stiffness. The horizontal axis (normalized strength) represent the ratio of the abutment yield strength to the strength of the other lateral resisting com-
ponents (columns in this case) in the bridge system. Increasing the strength reduced the ductility demands and increased the system forces, although the column forces remained similar due to yielding. The effect was more pronounced for short periods and high target ductility values. By comparing the increase in the total force between the longitudinal stiffness and strength parameters (figures 5.3(b) and 5.4(b)), it can be seen that the abutment strength parameter has a larger effect on the system forces than the stiffness parameters. Such behavior could be attributed to the increase in the target strength ratio of the abutment that attract more forces to the abutment. Such results confirm Mackie et al. [74] and Omrani et al. [101] observations. Both studies concluded that an increase in the abutment stiffness and/or strength reduce the displacement and have significant impact on bridges responses.
Figure 5.4: Abutment longitudinal strength as a dynamic parameter.

It is important to mention here that the minimum abutment stiffness ratio for the abutment sub-system, comparing to OpenSees results is 1.35 which is for ACST_SKJ (shear key-back wall-stem wall-wing wall CJ case). The minimum strength ratio for the abutment sub-system is 1.1 which is also for the same mentioned case (ACST_SKJ). By observing the SDOF system results shown in figures (5.3 and 5.4), the mentioned abutment stiffness and strength values provide the maximum difference for the displacement reduction and total force increase. Once again, such outcome
confirms the abutment subsystem findings that are the significant underestimation for the system forces and overestimation for the displacement demand.

Columns yielding in the transverse stiffness parameter study also led to negligible effect on the column forces. Figure (5.5) illustrates the effect of the abutment transverse stiffness on the responses. The horizontal axis (normalized stiffness) represent the ratio of the abutment stiffness to the stiffness of the other lateral resisting components in the bridge system. In the transverse direction, the other lateral resisting components include the columns and superstructure flexural rigidity in the transverse direction. The results were similar to the longitudinal stiffness findings. It can be seen that the system ductility decreases when the abutments’ behave elastically. However, early abutment yielding at low abutment stiffness (25-50% of the system stiffness), especially with yielding columns and high target ductility values, results in rapid increase in the system forces reaching up to its maximum strength. Such early absence of the forces (resisting components) that are required to reduce the ductility will induce counter effect by increasing the ductilities. On the other hand, despite columns and abutments yielding, increasing the abutment stiffness more than 50% of the system stiffness with low target ductility values provide sufficient resistance that recurs the ductility reduction. Such behavior reveals that synchronization of abutments and columns yielding, particularly at low abutments’ stiffness and high target ductilities, could have a substantial impact on the behavior.

In fact, such results indicate that adopting low abutment stiffness, with a range of 25-50% of the system properties (e.g., only the back fill stiffness/strength), as recommended by design codes would lead to displacements increase causing several problems such as unseating. Moreover, as shown in the subsystem results, abutments have an excessive underestimate resistance (structural components resistance), and its yielding would be due to excessive damage to its capacity protected components.
The effect of the abutment transverse resistance is shown in figure (5.6). The results showed a similar trend to the transverse stiffness parameters. However, it was observed that the strength parameter has less impact on the behavior. The low strength values have a minor effect on the ductility and no effect of the system forces. Such outcome could be attributed to the minor effect of the system mass and stiffness, and its corresponding abutment responses, on the system behavior. It should be remembered here that, in the transverse strength parameter, constant values were used.
for the generalized mass and stiffness of the SDOF system. Similar to the longitudinal direction, figure (5.5(a)) also show that abutment stiffness and/or strength in the transverse direction have larger effect on short period bridges and high target ductility values.

Figure 5.6: Abutment transverse strength as a dynamic parameter
5.6 Bridge Dynamic Analyses Results

5.6.1 The PSDM results

Twenty four values of each abutment parameter were considered in each analyses set (160 ground motion bin). As the selected abutment subsystem was California’s Murphys Grade Road (MGR) seated type abutment, same bridge will be considered in the bridge dynamic analyses. This was done to investigate the abutment subsystem findings and determine the effect of the abutment mentioned parameters and it’s modeling assumptions and simplifications on the bridge response.

Caltrans [20] ARS spectrum for soil type D with magnitude of 7.5 and PBA equal to 0.7g, with scale factor equal to 2, was used in the bridge dynamic analyses in order to insure inelastic and maximum bridge responses.

As observed in the SDOF system, it was found that the abutment parameters have negligible effect on the columns forces while it reduces the bridge displacement demands significantly. The PSDM results for the abutment parameters is shown in figure (5.7). In the figure, that shows the predicted bridge ductility versus the abutment parameters, it can be clearly observed that increasing the abutment stiffness or strength reduces the displacement ductility effectively especially for stiffness and strength values that is more than 200% of Caltrans [20] standard values. On the other hand, the disadvantage of the stiffness and strength enlargement is increasing the base shear which in turn will overload the abutment and piles causing undesirable damage or failure. Figure (5.8) shows an averaged total base shear for each abutment parameter (stiffness and ultimate strength in the longitudinal and transverse direction). It can observed from the figure the considerable increase in the base shear that results from the additional abutment stiffness and/or strength. The outcome of such underestimated forces, as shown in the abutment subsystem results, is shifting the critical components behavior such stem wall and piles. As a matter of fact, it will alter the response from
the elastically designed response to inelastic dominant response.

Figure 5.7: Bridge Ductility (PSDM results)
5.6.2 The Effect of the Longitudinal Stiffness

The columns and abutment forces for several values of the longitudinal stiffness are shown in figure (5.9). It can be noted the early engagement of the abutment in the response and the amount of force that have been transformed to the abutment. Also, it is evident that the abutment is resisting the forces in the high acceleration period of the ground motion and yielding due to the excessive forces in the boundary conditions. Another important observation is the absence of the abutment
contribution in the following region which could be attributed to the excessive propagation of the
gap, abutment yielding, and the transfer of the forces to the columns. Though, due to the displace-
ment reduction, it can be noted that the columns force is minimally affected by the stiffness change
(peak columns forces have close values for the specified stiffnesses). Figure(5.10) illustrates the
left and right abutment hysteresis for the selected longitudinal stiffness values. It can be seen that
the behavior of the abutment confirms the previous results that show the abutment failure. Also, it
is important to notice the increase in the abutment displacement, abutment’s yielding, gap propa-
gation due to the increasing in the abutment engagement and contribution to the bridge behavior.
Finally, figure (5.11) illustrate columns longitudinal and transverse displacements for the same
selected stiffness values. It can be observed from the figure the decrease in the columns displace-
ment due to the stiffness increase that stabilizes at high stiffness values. Also, the displacement
plots reveal the fact that despite the longitudinal displacement reduction, there is an increase in the
transverse displacement which shed light on aforementioned potential problems that could result
due to the unbalance between the longitudinal and transverse displacements. Due to the decrease
in the longitudinal displacement displacement, the transverse displacement will be significantly
larger than the longitudinal displacement that in turn will result in unbalance between the longi-
tudinal and transverse displacement and contradict the design codes’ floating bridge theory and
consistency between the displacements in both directions. Such gap between the two direction
displacements will concentrate the damage in one direction (transverse direction) causing several
potential problems such superstructure unseating or columns failure.
Klong = 100% Standard Value

Klong = 200% Standard Value

Klong = 370% Standard Value

Figure 5.9: Columns and Abutments longitudinal Forces
Figure 5.10: Abutment Hysteresis in the Longitudinal Direction
Figure 5.11: Column Displacements in the longitudinal and transverse direction
5.6.3 The Effect of the Longitudinal Strength

The effect of the abutment longitudinal strength on the bridge behavior is presented in this section. Figure (5.12) shows the columns and abutment forces for several abutment strength values. It is important to remember here that Caltrans [20] standard value for the passive pressure is 239 kPa. It can be observed from the results the significant impact of the abutment strength considering the enormous amount of force that were resisted by the abutment. Also, it could noticed from the figure that the abutment resistance represent more than 150% of the columns forces especially for the strength values that are larger than the specified standard value. The abutment hysteresis behavior is shown in figure (5.13). From this plot, it can be seen that the abutment is yielding for strength values around 150% the standard value and behave elastically (continues resisting the loads without failure) for strength values around 200% of the standard value which can be attributed to the extreme abutment strength. The impact of the abutment strength is also apparent in figure (5.14) that shows the columns displacements for the selected longitudinal strength values.

It was found that the abutment strength has a noticeable displacement reduction and that effect stabilizes for strength between 75% and 150% of the standard value. Such behavior agrees with the SDOF system results. In the SDOF results, the displacement reduction stabilize for longitudinal strength values that are more than 75% of the standard strength. Yet, the 3D analyses, as one of its advantages, unveil the effect of three directional system behavior that can be characterized by the difference between the longitudinal and transverse displacement that could lead to several potential problems as mentioned earlier. Such behavior could not be captured using the SDOF system. It should be mentioned here that, in the longitudinal direction analyses, the gap was assumed to be zero to avoid its effect on the behavior. Considering the gap in the analyses will delay the abutment effect (engagement) in resisting the force, accelerating columns yielding due to the absence of abutment’s contribution, reduce the forces that will be transfered to the abutment
and delay abutments’ yielding. Such effect could be useful because it accelerate plastic hinges formation that help dissipating the energy, though, it increases columns’ damage and in turn the possibility of columns’ failure.
Plong = 0.0

Plong = 76% Standard Value

Plong = 150% Standard Value
Figure 5.12: Columns and Abutments longitudinal Forces
Figure 5.13: Abutment Hysteresis in the Longitudinal Direction
Figure 5.14: Column Displacements in the longitudinal and transverse direction
5.6.4 The Effect of the Transverse Stiffness

Figure (5.15) shows the columns and abutments forces for the selected transverse stiffness values. It can be noticed from the figure the early engagement of the abutment and the rapid increase in the abutment forces with increasing the stiffness. Moreover, it can be noticed that the columns forces, especially for high stiffness values, represent about 50% of the abutment forces. Such behavior reveals the important impact of the abutment stiffness and the significant amount of force that will be mobilized to the abutment. The design codes recommend that shear keys should dissipate the energy that result from the superstructure impact with abutments in the transverse direction and no force should be transferred to the abutment’s components such as stem wall and piles. Yet, due the underestimated shear keys resistance that were observed in many studies, the additional forces will be transferred to the stem wall and piles leading to damage or failure in these components. Such consequence can also be observed in figure (5.16) that shows the abutment hysteresis behavior. In the figure, it can be clearly seen that the abutment experience inelastic behavior for several cycles and for all the illustrated stiffness values. As mentioned, this can be attributed to the early and significant contribution of the abutment.
Figure 5.15: Columns and Abutments Transverse Forces
Figure 5.16: Abutment Hysteresis in the Transverse Direction
Figure (5.17) shows the columns longitudinal and transverse displacement through time. It can be noticed that the abutment’s transverse stiffness parameter have a similar effect to the longitudinal stiffness. This effect can be characterized by the effective reduction in the transverse displacement and considerable difference between the longitudinal and transverse displacement that grows with increasing the stiffness. In the longitudinal direction, the full deck mass is contributing to the behavior while the more the abutment stiffness, the less the deck mass contribute in the response. Besides, comparing the longitudinal and transverse stiffness results, it can be noticed that the transverse stiffness has less impact than the longitudinal stiffness in reducing the displacement. To illustrate, it can be seen that the longitudinal displacement reduction due to the increase in the longitudinal stiffness is more significant than the transverse displacement reduction due to the transverse stiffness increase. Such response, as observes in the SDOF system results, could be attributed to the effect of the transverse displacement shape and generalized stiffness and mass that contribute to the response. In the longitudinal direction, the full deck mass is contributing to the behavior while the more the abutment stiffness, the less the deck mass contribute in the transverse direction response.
Figure 5.17: Column Displacements in the longitudinal and transverse direction

Displacement (m)

Ktrans = 100% Standard Value

Ktrans = 200% Standard Value

Ktrans = 370% Standard Value

Time (sec)

Displacement (m)
5.6.5 The Effect of the Transverse Strength

The final abutment parameter is the abutment ultimate strength. Figure (5.18) illustrates the columns and abutment forces with the increasing the abutment strength. As observed in the longitudinal stiffness parameters, the increase in the abutment strength increases the abutment forces and in turn the base shear except that in this case the abutment forces is more than 2.5 times the columns forces which can be attributed to the transverse resistance modeling configuration that characterized by elastic perfectly plastic material behavior comparing to elastic perfectly plastic gap material for the longitudinal resistance. Also, as can be seen in figure (5.19) that shows the abutment hysteresis, the abutment behaves elastically for most of the abutment transverse strength values which emphasize the drastic impact of such excessive forces on the piles and ground soil response. Finally, figure (5.20) shows the columns longitudinal and transverse displacement for several strength values. The figure shows reduction in the displacement with increasing the transverse strength, however, the transverse displacement in this case is still larger than the longitudinal displacement and that again could be attributed to the transverse displacement shape and participating mass.
Figure 5.18: Columns and Abutments Transverse Forces
Figure 5.19: Abutment Hysteresis in the Transverse Direction
Figure 5.20: Column Displacements in the longitudinal and transverse direction
5.7 Chapter Discussion

The study of the abutment parameters in the longitudinal and transverse direction agrees with the single degree of freedom system findings. It was found that, besides the columns yielding, the abutment has an early and significant contribution to the behavior. In addition, it was observed that the increase in these parameters reduces the displacement demand of the bridge structure and increases the amount of forces that will be mobilized to the abutment (boundary conditions) inducing inelastic behavior in the abutment components. It was also noticed that the abutment strength has higher impact on the behavior than the abutment stiffness. Such effect, as observed in the results, is more distinct and effective in short period bridges and higher target ductility values. Moreover, the results also confirm the abutment subsystem findings and unveil several disadvantages and potential problems that could arise as a result of these effects. It is important to point out here that, according to the abutment finite elements results in the longitudinal direction, the least abutment stiffness (ACSL_BKWJ) is more than 1.4 times the stiffness of OpenSees model and the abutment strength is more than 1.1 times the strength of OpenSees model. In the transverse direction, the least abutment stiffness (ACST_SKJ) is 85% of OpenSees model stiffness while the strength is more than 1.3 times OpenSees model strength. Despite that the previous limits show minimal difference in the total forces and displacement, yet, several important points should be noticed:

(i) There is considerable amount of forces that are mobilized to the abutments altering the load path and shifting failure mode leading to undesirable damage. (ii) in a design prose, it is a common assumption to model the abutments as roller to maximize the responses (for conservative design). Such assumption will substantially underestimated forces and overestimated displacements that in turn will intensify the gap between the actual and predicted response. (iii) In an assessment prose, adopting such results will lead to incorrect understanding to the bridge performance and false judgment regarding the components performance . Moreover, it should be remembered here that the analysis performed in the FEM is a nonlinear static analyses and considering a nonlinear
dynamic analyses in the FEM would show that the actual responses is even larger than the results obtained in the simplified dynamic analyses which can be attributed to the nonlinearity due to the loading cycles.

The consequence of the additional resistance that is available in the system and the considerable amount of forces that will be transformed to the abutment due to this resistance is altering the behavior and inducing inelastic and undesirable response in several components. As shown in the FEM results, the consequences of excessive amount of forces that were mobilized to the abutment and abutment’s high strength is unpredicted damage in several abutment’s components. These consequences include stem wall and wing wall damage in the longitudinal direction and stem wall, back wall, and wing walls damage in the transverse direction. Also, despite that such case is not considered in the this study, the mobilized force could overload the foundation system causing excessive foundation displacement, excessive deformation of the soil, or failure in the piles or ground soil as observed in several real events [145, 60, 122, 67]. The other possible case is when the abutment behave elastically due to its excessive strength, as observed in the longitudinal and transverse strength study in the MGR dynamic analyses, this resistance could effect the superstructure by causing failure or local damage in the girders due to the impact between the strong abutment and superstructure or girders unseating due to superstructure rotation. Moreover, in general, a reduction in the longitudinal displacement, for instance, will make the transverse displacement larger than the longitudinal displacement that will concentrate the damage in the transverse direction causing failure in the lateral resisting systems such as the columns due to the significant forces that will be propagated in these components.

Moreover, in seismic events and with excessive abutment’s stiffness and/or strength in the longitudinal direction, due to the considerable reduction in the longitudinal displacement, the transverse displacement will be significantly larger than the longitudinal displacement that in turn will result in unbalance between the longitudinal and transverse displacement and contradict the design codes’
floating bridge theory and consistency between the displacements in both directions. Such gap between the two direction displacements will concentrate the damage in one direction (transverse direction) causing several potential problems such superstructure unseating or columns failure.

The abutment parameters study unveil the actual load path and failure mode that should be used in bridges’ simplified models. It was found that the actual load path is that more than 50% and 75% of the load is passing through the abutments’ structural components in the longitudinal and transverse direction, respectively. Such load path shows that the back fill is not the only resisting element to the superstructure displacement and the abutment structural components have considerable contribution to the resistance. The back wall and shear keys sacrificial role, on the other mode, is not the actual abutment’s failure mode, according to the FEM results, the observed failure mode is inelastic behavior (damage) in several critical components in addition to the possibility of undesirable consequences such as excessive foundation displacement or ground soil or pile failure. The previous consequences, as mentioned, is due to the modeling simplifications and assumptions that are commonly adopted in bridges design and assessment procedures. Examples of these assumptions are the sacrificial role of the backwall and shear keys, misconception of the backfill as the only resisting element to the deck movement, and the neglect of several components contributions such the wing wall and phenomena such as the friction between the abutment and soil.

For the longitudinal direction, providing the CJ in ACSL_BJ, ACSL_BKJ, and ACSL_BKWJ significantly reduce the subsystem resistance and the latter configuration provide a potential back wall sacrificial behavior. Yet, there is still more than 10% resistance in the subsystem that could be attributed to the stem wall-back wall-wing wall reinforcement. In addition, the back wall did not break or fail for all longitudinal FEm and contribute to the behavior through the reinforcement.

Considering shear key CJ in the transverse direction reduce the resistance significantly, yet, de-
spite that we have stem wall-shear key-back wall CJ, no shear key-back wall reinforcement, and less shear key-stem wall reinforcement, there is still 30% available resistance in the subsystem. As a result, such load path and failure mode assumptions understate the significant impact of the components such as the shear keys contribution, stiffness, strength, configuration (brittle or ductile), and actual behavior leading to unpredicted response or failure.
6.1 Summary and Conclusions

An essential and critical problem area in bridges’ design and assessment procedures is the contradiction between the observed and predicted bridges’ response that would be ascribed to the deficiencies in the simplified models, that utilize one-dimensional elements and simple mathematical representation, which are commonly used in such procedures and the incapability of these models to capture the correct response. Other reasons for such shortcomings are the design codes assumptions regarding the load path and failure modes, hypotheses used in bridges’ components design which commonly adopted in the simplified models, and neglect of several bridge components’ contribution.

In this study, the simplified model’s deficiencies were identified in three levels that include bridge components study, bridge subsystems study, and full bridge system study. In the components study, several bridge components were studied that are a bridge pier, abutment back wall, abutment shear key, bridge T-girder, and elastomeric bearing pad. In the subsystem study, two critical subsystems were investigated that are abutment subsystem and columns bent subsystem. Finally, dynamic analyses of full bridge systems were performed to determine the effect of the abutment modeling simplification and assumed properties on bridges’ behavior. Two approaches were used in the full bridge study that are single degree of freedom (SDOF) approach and Probabilistic Seismic Demand Analysis (PSDA) approach. In the abutment parameters study, four parameters were investigated which are the abutment stiffness and ultimate strength in the longitudinal and transverse direction. The following conclusions may be derived from this research:

(1) The simplified models provide fair estimation for the components strength and overall response.
However, in some cases, they were not able to capture the correct component response, load path and failure mode, damage propagation and nonlinear behavior, and special cases response such as deep beam action. The finite element models (FEM) provided an good estimation of the components’ response. More importantly, the FEM were able to provide an excellent prediction for the components damage, failure mode, and nonlinear response which provide better understanding for the effect of each component on the response.

(2) The simplified models and the design codes assumptions for the load path, failure mode, and components behavior that are utilized in these models induce considerable underestimation for the subsystems’ response. In the abutment subsystem, it was found that the simplified models underestimate the longitudinal system resistance by 10-60%. Such underestimation can be attributed to the back wall sacrificial role assumption, neglect of several components’ contribution such wing and stem wall, and neglecting important phenomena such as back fill-wing wall friction. Considering the back wall CJ configurations reduced the resistance underestimation, significantly altered the response, and provided potential back wall sacrificial behavior. Yet, there is still more than 10% resistance underestimation.

In the transverse direction, besides the non-sacrificial shear key role, it was noted that the simplified models underestimate the system resistance by 70% for the shear keys monolithic connection configuration and 40% for the construction joint connection configuration. It was also observed that the wing walls transverse resistance, that is usually neglected in the simplified models represent about 20% of the total system resistance. The additional system resistance in both directions have a significant consequence on the response which can be characterized by shifting the load path and changing the failure mode causing inelastic behavior in several critical components which in turn may result in undesired and unpredicted damage or failure.

(3) In the bent cap subsystem, it was observed that, while the columns suffer most of the defor-
mation in the FEM, the cap beam in the simplified models undergoes severe torsional and flexural
deformation with acceptable columns deformation that contradicts the design codes assumptions.
Such behavior is due to the use of the one-dimensional elements and the infinitesimal connection
point between the cap beam and superstructure that causes excessive flexibility and deformation
concentration at this point. The consequence is shifting the cap beam failure mode from torsional
failure upon reducing the torsional stiffness to flexural failure when increasing the cap beam tor-
sional stiffness despite that neither represents the correct system response.

(4) Based on the SDOF system results, due to the columns yielding, increasing in the abutment
stiffness or strength have negligible effect on the columns forces while reduces the displacement
demand of bridge structures and increases the system forces, considerably. Such increase in the
forces, as observed in the results, will increase the amount of forces that will be mobilized to
the abutment (boundary conditions) inducing yielding and inelastic behavior in the abutment. A
potential consequence of such excessive force, as observed in several real events, is overloading
the foundation system causing excessive foundation displacement, excessive deformation of the
soil, or failure in the piles or ground soil.

(5) The full bridge dynamic analyses confirmed the SDOF system results. It verified the displace-
ment reduction, forces increase, and yielding and inelastic behavior of the abutment. The full
bridge dynamic analyses also provided better visualization to the resulted difference between the
displacement in the longitudinal and transverse direction. The increase in the gap between the lon-
gitudinal and transverse displacement could alter the failure mode leading to unpredicted response.
Some of the potential problems are columns failure, superstructure unseating, and abutment failure.
6.2 Recommendations for future work

The simplified models that employ mathematically based representations and one-dimensional elements fail to capture the correct response, and that is due to the element’s basic assumptions and load path and failure modes assumptions that are used in the elements’ calibration proses. The following recommendations can be made according to the research findings:

(1) Performing bridges’ dynamic analyses for several bridge structures to determine the sensitivity of the abutment parameters to bridges’ configurations such as bridge geometrical regularity, the number of spans, single/multi columns bent, shear keys ductility, and other design parameters. A direct relationship was observed between the abutment parameters and bridges’ configurations [101, 74].

(2) Developing SDOF systems and performing bridges’ dynamic analyses to determine the effect of the column bent parameters (cap beam torsional and flexural rigidity) on bridges’ response. Such study will reveal the capability of the simplified models and the one-dimensional spine elements, in particular, to reflect the cap beam and superstructure response in real events. Studies showed that the use of beam stick element to represent the bridge’s box-girder and bent cap induce excessive flexibility and deformation in the cap beam [84].

(3) Improving the current one-dimensional spine elements based on experimental studies to enhance its efficiency to capture responses such as beams high aspect ratio, shear deformation, and damage propagation, and multi-axial states of stress.

(4) The current zero-length elements are used model many components such shear keys, backwall, backfill and bearing pads. It is recommended to develop elements that are specialized in each component response especially for controlling and complicate responses such the shear keys. Developing new elements that provide better representation to the components response will introduce
higher confidence and superior efficiency for the simplified modeling and analyses techniques.

(5) Due to the drastic abutment impact on bridges behavior, it is recommended to conduct full-scale experimental tests for abutment subsystem to investigate the actual system behavior.

(6) The current design codes revisions should be revisited and revised to include other components contribution such as stem wall and back wall. It also recommended to provide the design practice with recommendations that ensure the proposed components behavior such as back wall-shear key, back wall-stem wall, and back wall-wing wall construction joints. A new and specific reinforcement details at the aforementioned interfaces should be also provided to ensure the assumed failure mode.
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