SEISMIC RETROFIT OF STEEL TRESTLE BRIDGES

Chad Nakamoto

and

Ian N. Robertson

UNIVERSITY OF HAWAII
COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

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In memory of my grandparents, Hideo and Shinobu Tanaka whose unconditional love and support have made my pursuit of a dream possible.
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Chapter 1: Introduction and Project Description

1.1 Introduction

From 1911 through 1913 five steel trestle bridges were built across deep ravines for the old Hamakua Coast Railroad, on the island of Hawaii (the names and locations of the bridges are shown in figure 1.1). The railroad “ceased operation as a result of damage to the bridges and other facilities from the 1946 tsunami” (Klein, Koob & Lee, 1985). The surviving bridges were repaired using salvaged members from bridges that were destroyed, and widened to accommodate two-lane highway traffic. “The Hawaii Belt Road (FAP 19) carries an average daily traffic of about 5000 vehicles” (Klein, Koob & Lee, 1985) along the Hamakua coast and has become an essential link between Hilo and the Kailua-Kona area on the Leeward coast of Hawaii.

The bridges are located in a highly corrosive environment caused by salt water spray, high humidity, and warm temperatures. Despite continual maintenance, the condition of the bridges has deteriorated over the years due to corrosion of the steel girders and trestles. Design and shop drawings for the reconstruction of the bridges in 1949 and 1950 for highway traffic did not include design for current seismic codes and standards. The Hawaii State Department of Transportation in conjunction with the Federal Highway Administration has allocated funding for the retrofit of these steel trestle bridges subject to 0.42g acceleration (design requirement specified by Hawaii State DOT), along with many others on the island of Hawaii.
Figure 1.1 Kapue and Hakalau bridge locations
1.2 Project Scope

The Hawaii State Department of Transportation has apportioned the seismic retrofit of various bridges on the island of Hawaii into several packages. KSF, Inc., a structural engineering consultant, was selected to prepare plans, specifications, and cost estimates for the retrofit construction of bridges in the vicinity of Papaikou. The Papaikou package includes the bridges Kapue, Kaieie, Kalaoa and Hakalau. Kapue and Hakalau are steel trestle bridges (shown in figure 1.1) and Kaieie and Kalaoa are reinforced concrete bridges (not shown).

1.3 Project Objective

The complexity of latticed members and laced columns, and their behavior under cyclic loading and inelastic demands are unfamiliar to most engineers. Latticed members are seldom used in modern steel structures and evaluating the capacity of built-up sections and their connections poses an interesting challenge to engineers involved in seismic retrofit design. The objective of this research is to provide engineers with basic design principles, analysis background, and a methodology in computer modeling to design the retrofit for a steel trestle bridge subject to a serviceability limit state. Serviceability is defined as the ability to transport emergency services vehicles immediately following an earthquake and to maintain normal daily traffic access.
1.4 Project Description

This report focuses on the retrofit measures of the two steel bridges, Kapue and Hakalau (figure 1.2 and 1.3 respectively). Specific issues in computer modeling, dynamic analysis, rocking analysis, and nonlinear static pushover analysis are addressed. The report includes structural evaluation of latticed member capacity and their connections (figures 1.4 to 1.7), a seismic assessment of the existing bridges and suggestions for future retrofit design. Bridge retrofit measures, designs and details for the project are reviewed and force-displacement plots of individual bridge bents and a sample set of calculations for Kapue bridge are provided in the appendices.
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Chapter 2: Structural Analysis

2.1 Linear Analysis

2.1.1 Types of Analysis

Linear elastic quasistatic analyses are used as “seismic bridge analysis tools to (1) predict or define stiffness characteristics of columns, bents, or frames modeled with effective linear elastic properties, (2) determine deformation and force response in the linear elastic response range for an equivalent quasistatic seismic load input, or (3) determine structural displacements for the inelastic response range under the static simulated seismic load assuming that equal displacement or equal energy principles can be derived” (Priestley, Seible & Calvi, 1996). Analytical tools for linear elastic bridge analyses range from hand calculations utilizing simple beam, frame, or truss models, to standard structural analysis programs ranging from basic 2-dimensional frame codes to general purpose 3-dimensional finite element models.

2.1.2 Static Analysis

The basic assumption inherent in elastic analysis is that the stiffness of the components that comprise the structure is independent of the magnitude of loading. This supposition provides an allowance for proportioning and superposition. Proportioning enables the response of the structure to a larger loading than that for which it has been analyzed to be calculated by scaling the results from the original analysis. Superposition allows the deflection and stresses that occur under a combination of loads to be computed
under each individual loading. These two basic tools, proportioning and superposition, are independent of loading history. This enables designers to determine demands on a structure regardless of load magnitude or sequential application.

2.1.3 Elastic Response Spectrum Analysis

The response spectrum is defined as “the maximum response of a single-degree-of-freedom system with damping to dynamic motion of forces, and therefore it depends on the characteristics of the system and on the nature of the ground motion” (Priestley, Seible & Calvi, 1996). It is a function of dynamic input and of the period of vibration of the system assuming a constant viscous damping and linear elastic response. A smoothed elastic response spectrum for 5% damping was used as a basic tool for regionalization maps and for the inclusion of the effects of local ground conditions. The response spectrum of a ground motion at a given site depends on the energy released at the source, the distance from the epicenter, the geography of the site, and the local soil conditions. The quantities “commonly studied in terms of response spectra are displacements, velocities, and accelerations, which can be expressed by absolute values (taken with respect to the ground condition before an earthquake) or relative values (taken with respect to the ground during an earthquake)” (Priestley, Seible & Calvi, 1996). In seismic design, absolute acceleration, relative displacement and velocity are of interest and commonly plotted in response spectra. Normalized response spectra curves typically plot the ratio of spectral acceleration over maximum ground acceleration (ordinate) versus period (abscissa).
2.2 Nonlinear Analysis

2.2.1 Types of Analysis

Elastic analysis is not a particularly accurate tool for use in predicting the failure of structures or the behavior of structures as they approach a limit state. Preceding actual failure, a structure exhibits significant nonlinear, inelastic behavior. In nonlinear analysis, stiffness is dependent on loading. As a result, both "proportioning and superposition become invalid and the history of application of load becomes critically important" (SEAOH-Hamburger, 1997). Three types of analyses discussed in the following sections that incorporate nonlinear characteristics are time history analysis, inelastic response spectrum analysis, and static pushover analysis.

2.2.2 Inelastic Time History Analysis

Nonlinear time-history analysis investigates the dynamic response of a structure to a sequence of individual time-dependent force pulses of length or integration step $\Delta t$. At the end of the time step the structure's properties are adjusted to reflect the new internal stress and strain states, and are then used during the next time step. Time-integration solution strategies range from conditionally stable explicit schemes to unconditionally stable implicit integration schemes, the main difference being the numerical stability of the solution. "Numerical integration schemes for the time domain can have problems with accuracy or period distortion as well as numerical stability when the integration time step $\Delta t$ is not small enough" (Priestley, Seible & Calvi, 1996). Due to the required complexity
in fully cyclic member characterization, nonlinear time-history analyses are typically limited to frame type models in two or three dimensions.

Nonlinear time-history analysis depends on the importance of multidirectional response quantification and on the ability to model the various nonlinear cyclic member characteristics. It "evaluates the response of the bridge model to a particular earthquake ground motion input, and since intensity, duration, frequency content, and spatial and time variation characteristics of the actual seismic event for a given bridge site are associated with a large degree of uncertainty, such a nonlinear time-history analysis can be used in bridge seismic design or assessment only when the model with all the structural parameter variations is exposed to not just one but a suite of representative ground motion inputs" (Priestley, Seible & Calvi, 1996). A suite of several different input ground motions derived from diversified potential earthquake sources and modified with variability in local soil conditions can be expected to provide a reasonable basis for design or assessment response quantifications. Nonlinear time-history analysis "combines both the demand side of seismic evaluation in the form of earthquake ground motion input and the capacity side in the form of fully cyclic nonlinear member characterization" (Priestley, Seible & Calvi, 1996) permitting seismic capacities and demands for the global bridge structure to be evaluated and compared simultaneously.

2.2.3 Inelastic Response Spectrum Analysis

If the structure responds to dynamic excitation with nonlinear behavior, the initial period of vibration and elastic equivalent viscous damping of the system are not sufficient
to obtain the maximum response. Inelastic response spectra imply that the acceleration response spectrum has been modified to account for ductile member actions that results in reliable structural displacement ductilities. Member forces obtained from these inelastic acceleration response spectra can then be directly considered as actual maximum member demands. The inelastic response spectrum "requires that (1) all elements and joints are detailed to achieve the postulated structural displacement ductility level, and that (2) it applies only to the determination of forces, not to velocities or displacements" (Priestley, Seible & Calvi, 1996).

2.2.4 Static Pushover Analysis

Static pushover analysis determines the sequence of inelastic deformations, the formation of local mechanisms, and the formation of a global collapse mode. The structural model is physically altered to reflect the occurrence of nonlinear or inelastic action in the form of a change in member stiffness or the introduction of a hinge mechanism. "Requirements for a pushover analysis are (1) a linear elastic structural model, (2) initial or conditioning loads, and (3) the characterization of all important nonlinear actions or events" (Priestley, Seible & Calvi, 1996). The results of a pushover analysis are ultimate deformation capacities of bents or frames, as well as inelastic deformation demands on local mechanisms, related to predetermined capacities of members and connections. Special-purpose programs such as SC-Push3D iterate for axial load effects, update the structural stiffness at each integration step, and track inelastic deformations.
2.2.4.1 Advantages Of Static Pushover Analysis

A comprehensive evaluation of a lateral system would require the execution of a series of nonlinear time history analyses of the structure subjected to a representative suite of earthquake ground motions. This evaluation procedure is very costly and unfeasible in many cases. Static pushover analysis is a relatively simple evaluation process and provides more realistic and comprehensive data than a linear elastic analysis of the structure. The pushover procedure provides information on many response characteristics that cannot be obtained from linear elastic static or dynamic analysis and applies equally to the evaluation and retrofit of existing structures as to the design of new ones.

2.2.4.2 Limitations of Static Pushover Analysis

The pushover analysis is based on static loading and cannot represent dynamic phenomena with a large degree of accuracy. It may not detect important deformation modes that occur in a structure subjected to severe earthquakes, and the procedure may exaggerate others. Inelastic dynamic response may differ significantly from predictions based on invariant or adaptive static load patterns, particularly if higher mode effects become important. Limitations are also imposed by load pattern choices, however, this influence is less consequential for bridge retrofit due to the bridge deck representing a concentrated mass at the structure's highest elevation.
Chapter 3: Computer Modeling

3.1 GTSTRUDL

3.1.1 General Description

GTSTRUDL is a computer software system that provides "a highly user-oriented, comprehensive, state-of-the-art, and integrated general purpose structural analysis and design information processing system as a practical structural engineering design tool" (Georgia Tech Research Corporation, 1996). It provides the engineer with the ability to specify characteristics of structural problems, perform analysis, reduce and combine results, perform design, and output information on a selective basis. In spite of its universal features, the multi-dimensional finite element program was primarily used to perform an elastic or linear analysis on the individual bents of the steel trestles. The pier towers for Kapue and Hakalau were modeled both 2 dimensional (figure 3.2) and 3 dimensional (figure 3.3) in the transverse and longitudinal directions with pinned boundary conditions at the foundation. A typical bent elevation is shown in figure 3.1. The individual bents vary in height as shown in figures 1.2 and 1.3.

The top nodes of the piers were slaved to a master node located at the geometric center to model the rigidity of the bridge deck. These nodes were fixed in translation (horizontal x-axis) for plane frames and slaved in translation (horizontal x-axis and z-axis) and rotation (about the vertical y-axis) for space frames. Member connections were modeled as pinned, and a static load was applied at the master node in the direction
parallel to the ground motion. In the bridge longitudinal direction, two displacements were specified, Hilo to Honokaa and Honokaa to Hilo, due to the differences in pier stiffnesses in each direction. The trestle's elastic stiffness, period and seismic response coefficient were then determined to calculate the structure's elastic spectral displacements.

![Diagram of a bridge structure](image)

**Figure 3.1 Typical bent elevation**

### 3.1.2 Input Parameters

Torsional section properties for all members were assumed to be zero and bending moment of inertia about the weak and strong axes of built-up members were computed.
Figure 3.2 2-dimensional view of GTSTRUDL computer model

Figure 3.3 3-dimensional isometric view of GTSTRUDL computer model
manually using the parallel axis theorem assuming zero shear lag. The STEEL TAKEOFF command was utilized to determine the total weight of the steel trestle towers comprised of a diverse selection of built-up latticed members. The computed mass of the structure was used to calculate the seismic response coefficient and the displacement demand on the piers.

Slaving of various nodes were conducted through rigid bodies defined by the TYPE RIGID command and the RIGID BODY INCIDENCES command. Rigid bodies are special purpose elements which are used to define several types of planar and special constraints between a master joint and one or more slave joints. The rigid plane element consisting of three degrees of freedom, \( u_x, u_y, \) and \( \theta_y \), was used for 3-dimensional modeling. The JOINT TIES command was used to impose constraint conditions between selected joints of a structure in 2 dimensions. A joint ties constraint between a master joint and one or more slave joints is characterized by a one-to-one equality between relevant global displacements at the master joint and the corresponding global displacements at the slave joint.

The SLAVE RELEASES command was used to change the status of specified joint displacement degrees of freedom from dependent to independent. When a joint is specified as a slave joint by either the JOINT TIES or RIGID BODY INCIDENCES commands, its relevant degrees of freedom become dependent on the master degrees of freedom according to the dependency relationships of the two commands. The dependency of particular slave degrees of freedom on the master degrees of freedom is
removed by releasing the appropriate slave force/moment degree of freedom components from the master-slave dependency relationships.

3.1.3 Programming Miscues

“Floating” master nodes were not allowed in GTSTRUDL through the JOINT TIES command. Nodes must be rigidly attached to the structure for compatibility and analysis purposes. The RIGID BODY INCIDENCES command defines an active rigid body element and identifies the joints to which the element is connected. It creates a rigid link between the master node and the corresponding slave nodes eliminating the requirement of beam or column elements, thereby permitting “floating” nodes.

3.2 SCPush3D

3.2.1 General Description

SC-Push3D is a 3D “finite element program implementing various section type models to define inelastic behavior of beams and columns” (SC Solutions, 1996). It performs a static pushover analysis of frame-type structures, linear and nonlinear, to determine the displacement capacity of existing bridges and buildings prior to retrofit design.

The pushover procedure is completed in two steps. First, a linear static analysis is performed utilizing only static loads. Second, a prescribed ultimate displacement to simulate seismic effects, referred to as displacement control, is imposed on the structure
monotonically and a nonlinear analysis is performed. The results from the first step are used as the initial conditions of the second step. The procedure is continued until the predefined ultimate displacement is attained.

Resembling the input of GTSTRUDL, 2-dimensional and 3-dimensional piers were modeled transversely and longitudinally with pinned supports. Nodes located at the piers apex were slaved in similar fashion with a specified displacement to simulate earthquake response.

3.2.2 Yield Surface

The lumped plasticity approach is adopted in SC-Push3D. It is assumed that the yielding takes place only in a concentrated plastic hinge located at the ends of each element. A four-dimensional force-moment interaction yield surface is used for the plastic hinge elements with 16 parameters provided to enable the user to define and adjust the shape of the yield function. Figure 3.4 shows a three-dimensional yield surface including

![Figure 3.4 Yield Surface](image-url)
interaction between axial force, F, and weak and strong axis bending, M_i and M_j. The fourth dimension included in the SC-Push3D yield surface is the torque applied to the section. For steel sections, a type 1 yield surface was specified providing interaction between four general section forces (axial force, weak and strong axis bending moments, and torque) and the effects of strain hardening or strain softening was neglected assuming a perfect elastic-plastic stress-strain relationship.

3.2.3 Input Parameters

Torsional rigidity was neglected in linear and nonlinear section properties and the elastic and ultimate tensile and compressive strength of steel cross sections were presumed equal. Generally, for structural shapes such as wide flange beams, the ratio $M_{\text{plastic}}/M_{\text{yield}}$ (shape factor) ranges from 1.08 to 1.14. A shape factor of 1.10 was arbitrarily chosen to project the ultimate yield moment, $M_{\text{ultimate}}$, or plastic moment, $M_{\text{plastic}}$, about the local y-axis and $z$-axis of the built-up cross section.

![Bilinear Approximation to Plastic Hinge Moment-Curvature Relationship](image)

$M_y$, $\phi_y = \text{moment, curvature at first yield}$

$M_n = \text{nominal flexural strength}$

$\phi_y = \frac{M_n}{M_y}$

$M_u = \text{ultimate flexural strength}$

$\phi_u = \text{ultimate curvature}$

Figure 3.5 Bilinear Approximation to Plastic Hinge Moment-Curvature Relationship
The post-yield stiffness $(EI)_y^p$ and $(EI)_z^p$ was determined from a bilinear approximation to the moment curvature relationship shown in figure 3.5. Elastic-plastic behavior was assumed and the post-yield stiffnesses were set equal to zero. SC-Push3D does not specify a standard set of units, therefore, consistent units should be used to describe all input parameters, except where noted in the design manual. US Customary units of kip-feet were used for the required input of the project.

### 3.2.4 Programming Miscues

SCPush-3D is designed with free-format input organized to encompass basic programming concepts and practices. The input, however, is not very user-friendly. Alphabetic characters must be uppercase, data items must be separated by commas with proper spacing between entries, and the use of decimal points for whole floating point numbers is required. Scientific exponential notation is allowed, i.e., the number 400000 may be represented as $4E5$, however, $4E+5$ is erroneous and will terminate the program. Also, there are no built-in unit conversions within SC-Push3D. Except where noted, the user must prepare the input in a consistent set of units. The output produced by the program will then conform to the same set of units.

The results of the pushover analysis relies resolutely on the exactness of the yield surface. It is imperative to minimize the step increment of the nonlinear analysis to insure that only one hinge forms at each step interval. SCPush-3D allocates a maximum of 100 step increments, which for large, complex structures is not sufficient. Despite the continued efforts of the author, his colleagues, and other structural consultants, the
number of increments can not be increased. SC Solutions, Inc., the developer and provider of the software, asserts that the original computer code can not be modified to incorporate a larger number of step increments.

SCPush-3D is ideally suited for analyzing simple plane frame structures. It performs very well under these circumstances where plastic hinging occurs in a few areas and compliance to the yield surface is observed. Computer modeling was performed initially for all steel trestles in 3-dimensions, however, input parameters of the designated yield surface were not conformed to. High multi-degree of freedom structures do not perform well using the program, with extremely long computational time sometimes in excess of 24 hours. At this juncture, the program was terminated and the structure remodeled as a 2-dimensional plane frame.

The preferred analysis tool is an inelastic plastic collapse mechanism analysis (pushover analysis), carried out on independent 3-dimensional stand-alone space frames considered to be completely separated or independent from adjacent frames. Complex member space frames eliminated this first option, leaving a 2-dimensional plane frame analysis as the only plausible solution. The modeling and computing time decreased significantly and the output represented the intended yield surface. Prescribed displacements were not limited to small displacement patterns and the number of increments exhibited trustworthy results. The only concern was that the simplicity of a 2-dimensional analysis may not capture torsional effects, out-of-plane forces, and secondary moments obtained from a 3-dimensional computer model. This phenomenon was
investigated and ratified to be trivial by comparing member forces of the smaller, simpler frames modeled in 3-dimensions to the 2-dimensional model. Further inquiry and discussion of the 2-dimensional and 3-dimensional modeling comparison is provided in the subsequent chapter.
Chapter 4: Discussion of Results

4.1 Types Of Analysis

The seismic evaluation of the steel bridges consisted of a lollipop analysis, locked up analysis, and rocking analysis. The three types of analysis rendered a full spectrum of force demands in the members and pier footings. Member element and joint forces corresponding to maximum pier displacements were compared to design capacities.

4.1.1 Lollipop

Lollipop analysis assumes the individual piers to respond independent of adjacent piers. Equivalent force-deformation relationships are idealized as simple mathematical formulations characterized by lumping or concentrating the bridge mass at the top of the pier in order to represent prototype bridge behavior. Steel trestle piers were modeled elastically in GTSTRUDL using the tributary span between bents to represent the mass of the bridge deck.

![Lollipop Analysis](image)

Figure 4.1 Lollipop analysis of individual bridge bent
The mass was represented by vertical dead loads applied at the top of trestle columns and a spectral analysis was performed on the trestle towers in the longitudinal and transverse directions to determine the displacement demand of the structure. The elastic displacements due to a designed earthquake loading were set equal to the inelastic displacements assuming equal displacement approximations. A simple schematic is presented in figure 4.1 representing a mathematical formulation of the lollipop analysis.

4.1.2 Locked Up

In the event that expansion joints became fused due to deterioration, corrosion, or faulty construction tolerances, an analysis was conducted locking the bridge in its entirety (figure 4.2). The individual pier stiffnesses determined from an elastic analysis, GTSTRUDL, were combined manually to calculate a total bridge stiffness, $K_{total}$, in the longitudinal direction only.

![ Locked up analysis of entire bridge ]

Transversely, the piers were assumed to act independently and were designed using the lollipop analysis. Displacements were calculated through spectral analysis using $K_{total}$ and
the bridge mass as input parameters. Inelastic displacements were assumed equal to the equivalent elastic response. Member forces were compared to the lollipop analysis output to determine governing structure force demands. The procedure rations higher seismic burden on short, stiff trestles and less emphasis on tall, slender piers.

4.1.3 Rocking

It has been observed after several earthquakes that “a number of structures had responded to seismic excitation by rocking on their foundation, and in some cases this enabled them to avoid failure” (Priestley, Seible & Calvi, 1996). Typically, rocking occurs in structures with large masses at some distance from the ground and comparatively narrow bases. The response of slender structures is usually “governed by the high overturning moment at the base, and if rocking and uplift is possible, this moment is then limited by the moment needed to lift the weight of the structure against the stabilizing moment due to gravity” (Priestley, Seible & Calvi, 1996). The internal forces and deformations throughout the structure will be limited correspondingly.

The advantage of a structure rocking stems from the efficiency of the mechanism in dissipating energy. The foundation impact on the soil beneath are assumed as purely inelastic collisions, and by equating momentum before and after impact a kinetic energy reduction factor may be obtained. The rocking behavior of bridge bents dampens the seismic force transmitted to the structure, however, it transfers the full load into half the columns and footings. Longitudinally, rocking is prevented by the rigidity of the bridge deck, limiting the analysis to examining the trestle towers in the transverse direction only.
The analysis was conducted by utilizing the inelastic base shear of the pier determined from a pushover analysis and applying it as a static load at the top of the structure. The resisting moment due to the self weight of the trestle towers and the weight of the pier footings exceeded the overturning moment corresponding to the base shear at each pier for Kapue and Hakalau bridge, thereby, eliminating the possibility of rocking behavior and its corresponding analysis in the transverse direction.

4.2 Results Of Analyses

<table>
<thead>
<tr>
<th>Bridge</th>
<th>Pier</th>
<th>Displacement (inches)</th>
<th>&quot;Lollipop&quot; analysis</th>
<th>&quot;Locked up&quot; analysis</th>
<th>&quot;Rocking&quot; analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Transverse</td>
<td>Longitudinal Hilo to Honokaa</td>
<td>Longitudinal Hilo to Hilo</td>
<td>Transverse</td>
</tr>
<tr>
<td>Kapue</td>
<td>5 (2D) +</td>
<td>2.73</td>
<td>~</td>
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* See figures 1.2 and 1.3 for bent locations and elevations
+ Bents 1–2 and 3–4 are identical to bents 5–6 for Kapue bridge
++ Bents 6–7 and 8–9 are identical to bents 4–5 for Hakalau bridge

Table 4.1 Displacement at top of bridge piers

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The displacements achieved by the aforementioned types of analyses are provided in table 4.1. The elastic displacements were computed using a single mode spectral analysis and the corresponding inelastic forces were determined through a pushover analysis. Discussion of single mode spectral analysis and pushover analysis are presented in the ensuing sections.

4.2.1 Single Mode Spectral Analysis

The single mode spectral analysis method is used to calculate the seismic design forces for bridges that respond predominantly in the first mode of vibration. Bridge bents were modeled in 2 and 3 dimensions with an arbitrary static load, \( P_{\text{applied}} \), applied at the location of the bridge deck to calculate the structure's elastic displacement, \( \Delta_{\text{elastic}} \), and corresponding stiffness, \( \frac{P_{\text{applied}}}{\Delta_{\text{elastic}}} \). The period of the piers equaled \( 2\pi/\omega \) where \( \omega \) is referred to as the natural frequency of the system, \( \omega=(k/m)^{1/2} \). The elastic seismic response coefficient \( C_s \), used to determine the design forces is given by the dimensionless formula:

\[
C_s=1.2AS/T^{2/3}
\]

where \( A= \) the Acceleration Coefficient equal to 0.42

\( S= \) the dimensionless coefficient for soil type I

\( T= \) the period of the bridge structure

The value of \( C_s \) need not exceed 2.5A.
The spectrum resulting from the elastic seismic response coefficient is given in figure 4.3. The response coefficient multiplied by the mass of the structure and effective peak ground acceleration in g's, 0.42, results in the equivalent static earthquake loading on the structure, $P_{\text{earthquake}} = C_s(\text{mass})(0.42)g$. The spectral displacement, $\Delta_{\text{spectral}}$, is calculated as the product $\Delta_{\text{elastic}}(P_{\text{earthquake}}/P_{\text{applied}})$. The structure demand is determined by analyzing member forces from a nonlinear static pushover analysis at the spectral displacement assuming behavior based on equal maximum deflections.

![Graph](image)

**Figure 4.3 Normalized Response Spectra**

4.2.2 Static pushover Analysis

The results of the elastic analysis, nonlinear analysis and spectral demand are represented graphically as force-displacement plots in figures 4.4 to 4.10. Figure 4.4
investigates the legitimacy and exactness of the SC-Push3D nonlinear analysis curve by means of a manual pushover calculation. The procedure was conducted in GTSTRUDL, modeling the pier as a plane truss and applying a static load at the location of the bridge deck. The load was increased manually until the first member yielded. The element was extracted from the structure and replaced by forces representing its yield strength at the end nodes. The procedure was repeated until global collapse or structure instability was attained. Graphical representations of the two curves compare favorably instilling confidence in the SC-Push3D program’s analytical methods and tools.

The intersection of the GTSTRUDL elastic analysis and the spectral demand represents the spectral displacement and elastic force demand, $P_{\text{elastic}}$, of the structure or pier. The normalized response spectra was converted to coordinates of equivalent static earthquake load, $P_{\text{earthquake}}$, and spectral displacement, $\Delta_{\text{spectral}}$, by recognizing that for all values of peak spectral response:

$$\Delta_{\text{spectral}} = \left(\frac{g}{4\pi^2}\right)SA(T^2)$$

where $\Delta_{\text{spectral}} = \text{spectral displacement, in inches}$

$T = \text{period, in seconds}$

$SA = \text{spectral acceleration, in g's}$

$g = \text{gravity constant (386 in./sec.}^2\text{)}$
The slope of the GTSTRUDL graph is linear, as it should be for elastic design, and represents the elastic stiffness of the pier. The initial slope of the SC-Push3D nonlinear analysis force-displacement curve is nearly identical to its elastic analysis counterpart. It represents the linear portion of the structure before yielding occurs. The inelastic force demand of the structure, $P_{\text{inelastic}}$, is the force corresponding to the spectral displacement on the SC-Push3D curve.

A three dimensional frame trestle generally consists of two plane frame four column bents connected by longitudinal frames. The equivalent seismic loading, or base shear, for the two dimensional transverse direction are therefore typically one half that of the three dimensional case. In a similar way, the 3-dimensional model has a transverse stiffness twice that of its 2-dimensional counterparts due to the 3-dimensional model engaging 2 plane frames to resist the earthquake load. Figures 4.4 and 4.5 illustrate these differences.

The force-displacement plots of Hakalau pier 12 and 13 are shown in figures 4.6 through 4.8. The pier was of particular interest because of its irregular stiffness in the longitudinal and transverse directions. Two dimensional plots in the transverse direction of piers 12 and 13 are represented in figures 4.6 and 4.7, respectively. The spectral displacement is greater for the taller pier, bent 12, than it is for bent 13 due to the difference in relative stiffnesses. The transverse spectral displacement of the three dimensional trestle shown in figure 4.8 is between the displacements for the individual bents 12 and 13 as would be expected. The inelastic force demand for the 3-dimensional
Figure 4.4 Force-Displacement plot: Kapue pier 5 transverse direction (2 dimensional)

Figure 4.5 Force-Displacement plot: Kapue pier 5~6 transverse direction (3 dimensional)
Figure 4.6 Force-Displacement plot: Hakalau pier 12 transverse direction (2 dimensional)

Figure 4.7 Force-Displacement plot: Hakalau pier 13 transverse direction (2 dimensional)

Figure 4.8 Force-Displacement plot: Hakalau pier 12~13 transverse direction (3 dimensional)
Figure 4.9 Force-Displacement plots: Hakalau pier 12~13 longitudinal direction

Figure 4.10 Force-Displacement plots: Hakalau pier 4~5
analysis is slightly larger than the combined forces from the individual 2-dimensional bent analysis due to the added stiffness provided from out of plane (longitudinal) members.

In figure 4.9, force-displacement plots for 2 and 3-dimensional analyses of Hakalau bridge pier 12-13 are portrayed in both longitudinal directions, Hilo to Honokaa and Honokaa to Hilo. The hinging sequence of frames depends on the direction of the push displacement. In the direction of Hilo to Honokaa (figures 4.9a and 4.9c), the structure is less stiff than pushing in the opposite direction (figures 4.9b and 4.9d) as evidenced by the slope of the elastic force-displacement plot. Attention should also be directed to the force demand on the structure. The 3-dimensional model includes all four plane frames in the longitudinal direction while the 2-dimensional model only represents one frame. Hence the base shear in the 3-dimensional plot is 4 times the value in the 2-dimensional plot. This occurrence is explained by the number of plane frames, four, in the longitudinal direction represented in the 3-dimensional model. Also, comparison should be made between the transverse and longitudinal force-displacement plots (figures 4.6 through 4.9). The simplicity of the longitudinal frames results in hinging in relatively few locations. This results in an almost perfect elastic-plastic response of the bridge pier. In contrast, the hinging sequences and number of hinging locations differ in the transverse frames. The added number of members and connections in the transverse direction results in a more gradual progression of lateral response. The SC-Push3D curves shown in figures 4.6 through 4.8 reflect this behavior.
Static pushover analyses were not performed in 3-dimensions for the taller piers of Hakalau bridge due to the constraints and limitations of SC-Push3D. The number of members, connections, and hinging locations exceeded the maximum step increments of the program. Figure 10 illustrates the lateral response of Hakalau’s tallest pier in the transverse and longitudinal direction using 2-dimensional models. The pier was very flexible in both directions due to the height of the structure. In the transverse direction, hinging occurred in more distinct locations as evidenced by a close relationship between the force-displacement plot and pure elastic-plastic behavior. Longitudinally, the frame behaves elastically since the intersection of the spectral demand and the SC-Push3D nonlinear analysis occurs on the linear slope portion of the curve.

The simplicity of 2-dimensional modeling does not capture the 3-dimensional stiffness of the trestle tower when the plane frames comprising the structure have different stiffnesses. Kapue bridge bents 1~2, 3~4, 5~6 and Hakalau bridge bents 4~5, 6~7, and 8~9 are symmetrical both longitudinally and transversely. The relative stiffness of plane frame modeling in 2-dimensions will be proportional to the number of bents provided longitudinally or transversely in 3 dimensions. Piers located at the ends of Hakalau bridge on the sloping terrain do not benefit from geometric symmetry and uniformity. In lieu of 3-dimensional modeling, piers were modeled in 2-dimensions in SC-Push3D and pushed to the modal spectral displacements of the 3-dimensional structure modeled in GTSTRUDL. Internal forces and deformations computed at this displacement provided an estimate of the strength and deformation demands on the structure.
4.3 Member Strength Capacity

4.3.1 Latticed Members

Information on latticed steel members is limited because laced columns are rarely used in modern bridge construction. A task group, "composed of university professors and Caltrans engineers, was established in September 1994 to address the issue of evaluating the capacity of latticed members of the San Francisco-Oakland Bay Bridge" (Altman, Fukushima & Hamoud, 1995). The main objective was to "provide engineers with basic design and analysis background reflecting the design philosophy and engineering practice employed by builders of the great steel bridges over a half century ago" (Altman, Fukushima & Hamoud, 1995). Figure 4.11 shows the basic components of latticed members.

![Diagram of latticed members](image)

Figure 4.11 Basic components of latticed members

The "design of latticed columns is influenced by global buckling, buckling or yielding of specific local segments of the member, buckling of lacing bars and lattice
frame, and the torsional buckling or distortion of the cross section” (Altman, Fukushima, Hamoud, 1995). Figure 4.12 illustrates the different modes of buckling.

![Diagram of buckling modes](image)

**Figure 4.12** Buckling of latticed members

Additional consideration should be given to shear on latticed members and effective length factors. Although lacing bars are designed to take shear stress equal to 2% of the total compressive stress in the member, it is generally assumed that the function of the lattice work is to assure integral action of the solid longitudinal segments and to hold the various parts parallel and the correct distance apart. The lacing needs to be properly spaced in the latticed network to insure that the slenderness ratio \( L/r \) of the individual elements or segments between the lacing connections is less than the slenderness ratio \( KL/r \) of the entire column. When the critical stress of a segment between lacing connections of the laced column or member (shown as segment a-b, figure 4.13a) is smaller than that of the column as a whole, the column will fail due to local buckling of the segment, which in turn causes the entire column to fail (figure 4.13b).
Figure 4.13 Local buckling between single lacing connections

Modeling the latticed element in GTSTRUDL and deferring the details of the calculation, critical buckling of latticed members was governed by global buckling. Shear and effective length factors have a small influence on member strength capacity.

4.3.2 Column Elements

Built-up column members were analyzed for the following standards: (1) Satisfy stability interaction criteria, (2) Satisfy yielding interaction criteria, (3) Insure local buckling does not occur, (4) Insure overall buckling does not occur in the strong direction. Additional reference to items 2, 3, and 4 is made in a sample set of structural calculations for Kapue bridge in Appendix B.
4.3.3 Beam Elements

Beam elements were modeled as truss elements and investigated for allowable axial force and local buckling in compression. Lateral torsional buckling was satisfied by the b/t ratio of local buckling. Appendix B provides additional reference and calculations.

4.3.4 Diagonal Elements

Tension elements were designed for stresses equal to 36 ksi excluding strain hardening. An ultimate strain of 0.15 was designated as the member limit state. Strains were calculated at less than 0.01.

4.4 Connection Strength Capacity

4.4.1 Member Connections

Design of connections is a controversial issue both from a demand analysis standpoint and from a capacity analysis perspective. The initial debate arises when deciding if the connection should be considered pinned as it was originally designed, or fixed which may more closely represent its behavior. The “question of considering a member as a pinned-ends column or a fixed-ends beam-column was put to the ASCE committee on Design of Steel Building Structures a few years ago” (Altman, Fukushima & Hamoud, 1995). The committee, after much deliberation, suggested that the designer can model trusses as either frame structures (rigid joints, beam-column members), or as
trusses (pinned joints, axially loaded members). Both assumptions are acceptable if the members and connections are proficient at carrying forces that result from such assumptions.

The analysis for seismic retrofit modeled steel bents as frame structures. Beam elements were modeled as pinned end members to accommodate the semi-rigid connections of the bridge structure. “Studies of cyclic behavior of riveted double angle connections have shown that these connections exhibit ductile behavior and flexibility (Shen & Astaneh, 1993). The extraordinary ductility of these riveted angles is attributed to the ductile bending of the outstanding angle legs of angles and to a lesser extent to the tension and bending ductility of the rivets.

Shear failure of rivets was examined at member ends for connection capacity. “Laboratory testing of rivets in direct shear via samples taken from the Hakalau bridge confirm the strength of rivets in shear to be 75% of its material tensile strength” (Klein, Koob & Lee, 1985). Supplementary review and commentary is provided in Appendix B.

4.4.2 Anchor Bolt Connections

The construction documents for the original railroad bridges were not available for review and the design drawings for the reconstruction to accommodate highway traffic did not specify the type and material strength of the anchor bolts. A307 bolts were assumed, yielding an ultimate tensile strength of 60 ksi. Other assumptions made were that the anchor bolts threads are in the shear plane and frictional resistance and corrosion do not
increase shear strength capacity. Design equations and calculations are also provided in Appendix B.
Chapter 5: Retrofit Required

5.1 Kapue Bridge

Bent symmetry in the transverse and longitudinal directions minimized the retrofit scheme of the bridge structure. Rocking of the piers do not occur and member forces compared favorably using the lollipop and locked up analysis. The capacity of all trestle elements and connections exceeded the force demand of the earthquake loading eliminating the need to retrofit the steel members or riveted gusset plate connections. Maximum pier displacements of 3.5 inches were less than the available seat width of 9 inches. Single cable restrainers, however, were installed across expansion joints at midheight of the plate girder web to restrict lateral movement and unseating, and seat extenders were provided at the pier expansion joints and at both abutments to further insure proper seating width. Theoretically, cable restrainers and seat extenders were not required by design. Their function was to provide a margin of safety against error tolerances in analysis procedures, unforeseen forces of nature exceeding expected ground accelerations, and the present condition (actual capacity) of the bridge. Figures 5.1 through 5.3 are designed retrofit details presented to the State of Hawaii by KSF, Inc.
Figure 5.2 Kapue bridge - Seat extender and type I cable restrainer
Figure 5.3 Kapue bridge - Seat extender at girder expansion joint
5.2 Hakalau Bridge

Height variation of the piers and the overall length of the bridge resulted in the need for additional retrofit to that designed for Kapue bridge. The anchor bolts in the pedestal footings of pier 12~13 were inadequate in shear due to the high stiffness of the trestle tower. Bridge analysis in the locked up position showed pier 12~13 resists a high percentage of the total seismic load, whereas, equal transference of load prevails at the nearly identical Kapue bents. The pier footings and base plate connections were encased in concrete to consolidate and transfer the shear from the column members to the foundation beneath. Longitudinal bumpers were also placed at both end abutments to prevent excessive horizontal deflection. The bumpers, attached to the steel plate girders, consist of built-up steel sections with a flat steel plate at the end designed to engage the abutment wall upon movement. Dual cable restrainers and seat extenders at expansion joints and at both abutments were installed to compensate for the large bridge displacements. Displacements at pier expansion joints and abutments exceeded available seat widths. The cable restrainers, seat extenders, and longitudinal girder bumpers were all used simultaneously to prevent the plate girders from becoming unseated. Details of the retrofit are provided in figures 5.4 to 5.9.
Figure 5.4 Hakalau bridge - Seat extender at abutment
Figure 5.5 Hakalau bridge - Seat extender and type III cable restrainer
Figure 5.6 Hakalau bridge - Seat extender at girder expansion joint
Figure 5.7 Hakalau bridge - Longitudinal girder bumper at abutment
Figure 5.8 Hakalau bridge - Footing retrofit and reinforcement
Figure 5.9 Cable restrainer details
Chapter 6: Conclusion

6.1 Overview

Although inelastic time-history analysis is clearly the most sophisticated method available for assessing bridge performance under seismic excitation, there are uncertainties in application and assessment procedures. There are relatively few computer programs "capable of realistic three-dimensional non-linear modeling, including interaction between biaxial inelastic response of columns, interaction of flexural and shear strength, modeling of joint characteristics, and modeling of degrading performance in terms of negative post-yield stiffness and degraded hysteretic response characteristics" (Priestley, Seible & Calvi, 1996). Without such modeling capabilities, there is little point in pursuing the complexities of inelastic time-history analysis.

Inelastic time-history analysis should be the emphasis for evaluation procedures of the future. The approach is currently deemed unfeasible with exception of unique problems and circumstances. Static pushover analysis fulfills the objective of performing an evaluation process that is relatively simple but more realistic and comprehensive than one based on a linear elastic analysis of the structure. The procedure accounts, in an approximate manner, for the redistribution of internal forces occurring when the structure is subjected to inertia forces that can no longer be resisted within the elastic range of structural behavior. It provides information on many response characteristics that cannot be obtained from a linear elastic analysis or dynamic analysis.
Static push analysis performed on individual piers can be used to determine the inelastic force-displacement response of the structure. The force demands on each bent are derived from the spectral displacements determined from an elastic modal analysis assuming equal maximum elastic and inelastic displacements. Member and connection capacities are calculated and compared to the corresponding force demands.

6.2 Recommendations

The purpose of carrying out a seismic assessment analysis of an existing bridge is to determine the level of risk associated with loss of serviceability, severe damage, or collapse. With this risk quantified, rational decisions can be made as to whether the bridge should be retrofitted or replaced, or to accept the risk and leave the bridge in the existing state. There has been a tendency in the past to use rather rudimentary analytical tools for this phase of analysis. It is imperative that engineers apply more analytical effort to reduce the seismic retrofit cost. Results of increased analytical effort are very cost-effective in identifying where retrofit cost savings can safely be made.

The search for a more rational and transparent design process will remain an issue for debate and further research in years to come. Design will always be a "compromise between simplicity and reality; with the recognition that reality is very complex and uncertain in imposed demands and available capacities, and simplicity is a necessity driven by limited fees and the limited ability to implement complexity with commonly available knowledge and tools" (SEAOC-Krawinkler, 1996). Static pushover analysis is not a final
solution to design and analysis problems, but it is a significant step forward to including consideration of inelastic response characteristics in bridge behavior.
References


Appendix A

Force-Displacement Plots of Kapue and Hakalau bridge piers
Figure A.1  Force-Displacement plot: Kapue pier 5~6 longitudinal direction (2 dimensional)

Figure A.2  Force-Displacement plot: Kapue pier 5~6 longitudinal direction (3 dimensional)
Figure A.3 Force-Displacement plot: Kapue pier 7 transverse direction (2 dimensional)

Figure A.4 Force-Displacement plot: Hakalau pier 1 transverse direction (2 dimensional)
Figure A.5 Force-Displacement plot: Hakalau pier 2 transverse direction (2 dimensional)

Figure A.6 Force-Displacement plot: Hakalau pier 3 transverse direction (2 dimensional)

Figure A.7 Force-Displacement plot: Hakalau pier 2-3 transverse direction (3 dimensional)
Figure A.8 Force-Displacement plot: Hakalau pier 2-3 longitudinal direction
Hilo to Honokaa (2 dimensional)

Figure A.9 Force-Displacement plot: Hakalau pier 2-3 longitudinal direction
Hilo to Honokaa (3 dimensional)
Figure A.10 Force-Displacement plot: Hakalau pier 2–3 longitudinal direction Honokaa to Hilo (2 dimensional)

Figure A.11 Force-Displacement plot: Hakalau pier 2–3 longitudinal direction Honokaa to Hilo (3 dimensional)
Figure A.12 Force-Displacement plot: Hakalau pier 10 transverse direction (2 dimensional)

Figure A.13 Force-Displacement plot: Hakalau pier 11 transverse direction (2 dimensional)
**Figure A.14** Force-Displacement plot: Hakalau pier 10~11 longitudinal direction  
Hilo to Honokaa (2 dimensional)

**Figure A.15** Force-Displacement plot: Hakalau pier 10~11 longitudinal direction  
Honokaa to Hilo (2 dimensional)
Appendix B

Sample Structural Calculations for Kapue bridge
A continuous load path from the concrete bridge deck to the foundation was analyzed for member/connection capacity versus seismic demand to insure proper load transfer. The following itemizations outline the procedure employed for retrofitting a steel trestle bridge.

I. Concrete deck to steel plate girders
   A. Shear hangers in concrete deck
   B. Welded connection to top of steel plate girders

II. Cross framing between steel plate girders
    A. Members
       1. Diagonals
       2. Columns (vertical elements)
       3. Beams (horizontal elements)
    B. Connections
       1. Rivets
       2. Welds
       3. Bolts

III. Connection between bottom of steel plate girders to top of trestle columns
     A. Rivets
     B. Bolts

IV. Steel trestle towers
    A. Members
       1. Diagonals
       2. Columns
       3. Beams
    B. Connections
       1. Rivets
       2. Welds

V. Steel trestle Base
    A. Connection
       1. Anchor Bolts
       2. Steel base plate
    B. Foundation
       1. Sliding resistance
       2. Uplift forces
       3. Overturning moment
       4. Bearing pressure
Steel Bridge mass (weight):

Concrete deck: \(0.160(8/12)(24.5) = 2.6 \text{ K/ft}\)

Sidewalk (2): \(0.160(8/12)(14) = 1.5 \text{ K/ft}\)

Railings (2): \(1 \text{ K/ft}\)

AC pavement (3") : \(0.150(3/12)(25) = 0.95 \text{ K/ft}\)

Steel plate girders: \(0.490(1.5/12)(6)(4) = 1.5 \text{ K/ft}\)

Continuous angles: \(0.0374(16) = 0.60 \text{ K/ft}\)

Cross frame: \(0.0085(184)(1/7) = 0.25 \text{ K/ft}\)

Connections: \(0.25 \text{ K/ft}\)

Miscellaneous: \(0.25 \text{ K/ft}\)

Total weight: \(8.9 \text{ K/ft} = 9 \text{ K/ft}\)
Analysis:

Steel Trestle Piers:

~Steel Structures Design and Behavior Second Edition, Chapter 12

Design requirements:

1. Tension members - diagonals
   A. Member stresses less than or equal to 36 ksi.
   B. Member strains less than or equal to 0.15

2. Compression members - columns
   A. Satisfy stability interaction criteria.
   B. Satisfy yielding interaction criteria.
   C. Insure local buckling does not occur.
   D. Insure overall buckling does not occur in strong direction.

3. Compression members - beams
   A. Satisfy allowable axial force.
   B. Insure local buckling does not occur.

Stability Interaction Criterion:

\[ \frac{P}{P_a} + \frac{C_m}{(1-P/P_e)M/M_y} \leq 1.0 \]

\[ P = \text{computed axial force} \]
\[ P_a = \text{allowable axial force} \]
\[ = (1-(K_l/r)^2/(2C_c)^2)F_yA \]
\[ P_e = \text{Euler buckling load} \]
\[ = \pi^2E/(K_l)^2 \]
\[ C_c = (2\pi^2E/F_y)^{0.5} \]
\[ C_m = \text{Moment magnification coefficient (factor)} \]
\[ M = \text{computed moment} \]
\[ M_y = \text{nominal moment strength (FySx)} \]

Yielding Interaction Criterion:

\[ \frac{P}{P_y} + \frac{M}{M/y} \leq 1.0 \]

\[ P_y = \text{yield load (FyA)} \]

Local Buckling:

\[ \frac{b}{t} \leq 76/(F_y)^{0.5} \]
Analysis:

Strength of Trestle structure connections:
~Steel Structures Design and Behavior Fourth Edition

Design assumptions:

1. Minimum tensile strength of weld is 70 ksi.

Rivets:

Shear capacity (stress) of rivet = .75 x (tensile strength of rivet steel)

= .75 (51 ksi)

= 38.25 ksi

Area of 7/8" rivet = \( \pi \left(\frac{7}{8}/2\right)^2 \) = .60 in\(^2\)

Shear capacity (force) of 7/8" rivet = 38.25 x .60 = 23 kips

Fillet welds:

\[ R_{nw} = te(0.60F_{Exx}) \]

\( R_{nw} \) = design strength per unit length of fillet weld
\( te \) = effective throat dimension
\( F_{Exx} \) = tensile strength of electrode material

\[ R_{nw} = \frac{5}{16}(.707)(.60)(70) = 9.28 \text{ kips/inch} \]

Note:

Tension elements

1. Zero end moments
2. Design connections for pure shear

Compression elements

1. Small end moments
2. Design connections for shear and bending
Analysis:

Strength of anchor bolt connection at tower base:
~Steel Structures Design and Behavior Fourth Edition

Design assumptions:

1. Tensile strength of A307 bolts is 60 ksi.
2. Threads of A307 bolts is in shear plane.
3. Frictional resistance and corrosion do not increase shear strength capacity.

\[(\frac{R_{UT}}{R_{NT}})^2 + (\frac{R_{UV}}{R_{NV}})^2 \leq 1.0\]

\( R_{UT} \) = tension load on bolt

\( R_{UV} \) = shear load on bolt

\( R_{NT} \) = design strength of bolt in tension alone

\[ = Fu(.75)Ab \]

\[ = 93.2 \text{ kips (1-5/8" diameter anchor bolt)} \]

\[ = 141.3 \text{ kips (2" diameter anchor bolt)} \]

\( R_{NV} \) = design strength of bolt in shear alone

\[ = .62Fu(m)(.75)Ab \]

\[ = 57.8 \text{ kips (1-5/8" diameter anchor bolt)} \]

\[ = 87.6 \text{ kips (2" diameter anchor bolt)} \]

\( Fu \) = tensile strength of bolt material

\( Ab \) = gross cross-sectional area across the unthreaded shank of the bolt

\( m \) = the number of shear planes participating
Rocking analysis
Kapue Pier 5
Transverse direction

Determine P load for overturning about point A

\[ P = 111(22.7) + 111(23.7) + 111(17.2) + 111(8.8) + 100(40.8) + 100(31.8) + 16(20.4) \]

\[ P = 340 \, k \]
Kapue Pier 5-6
Longitudinal direction
Kapue Bridge
Pier 5~6

Diagonal elements (tension members)

1A. Member stresses are less than or equal to 36 ksi for all tension elements in the transverse and longitudinal direction by inspection of SCPUSH-3D member forces.

1B. Member strains are less than 0.15 for all tension elements in the transverse and longitudinal direction by inspection of SCPUSH-3D member forces.
Kapae Bridge
Pier 5-6

Column elements (compression members)

2A. Check stability interaction criterion.

\[
P / P_a + C_m / (1 - P / P_e) * M / M_y \leq 1.0
\]

\[
\frac{357}{607} + \frac{0.31}{(1 - 0.31)} \cdot \frac{389}{19.44} = 0.66
\]

2B. Check yielding interaction criterion.

\[
P / P_y + M / M_y \leq 1.0
\]

\[
\frac{357}{635} + \frac{389}{19.44} = 0.76
\]

2C. Check local buckling.

Built-up column section will not local buckle.

2D. Check overall buckling.

\[
P / P_a \leq 1.0
\]

\[
\frac{357}{607} = 0.59
\]
Kapue Bridge
Pier 5~6

Column elements (compression members)

~Critical rigid frame element is element 162 (transverse direction)

\[ P = 357 \text{ K} \]
\[ M_1 = -23.2 \text{ K'L} \]
\[ M_2 = 32.4 \text{ K'L} \]
\[ L = 22.3' \]
\[ A = 17.64 \text{ in}^2 \]
\[ r_x = 5.04'' \]
\[ r_y = 5.04'' \]
\[ K_{L/r_x} = 0.7(22.3)(12)/5.04 = 37.2 \]
\[ K_{L/r_y} = 0.7(22.3)(12)/5.04 = 37.2 \]

\[ P_{ax} = (1 - (K_{L/r_x})^2/(2(Cc)^2))FyA = (1 - (37.2)^2/(2(126)^2))36(17.64) = 607 \text{ K} \]

\[ P_{ax} = (1 - (K_{L/r_y})^2/(2(Cc)^2))FyA = (1 - (37.2)^2/(2(126)^2))36(17.64) = 607 \text{ K} \]

\[ P_x = \pi^2EI/(K_{n,L})^2 = 2643 \text{ K} \]

\[ P_y = FyA = 36(17.64) = 635 \text{ K} \]

\[ M_y = S_{x_{max}}Fy = 54(36) = 1944 \text{ K'} \]

\[ C_m = 6 - 4(M_1/M_2) = 0.31 \]
**Kapue Bridge**

**Pier 5-6**

**Beam elements (compression members)**

~ Critical member truss element is element 129 (transverse direction).

3A. Determine allowable axial force (slender element).

\[
Pa = \left(1 - \frac{(KL/r)^2}{(2Cc^2)}\right)FyA(Qs)
\]

\[
KL/r = \frac{(1.0)(0.7\times12)}{1.7} = 75.5
\]

\[
Cc = (2\pi^2E/Fy)^{0.5} = 126
\]

\[
Fy = 36 \text{ ksi}
\]

\[
A = 4.18 \text{ in}^2
\]

\[
Qs = .99
\]

\[
Pa = \left[1 - \frac{(75.5)^2}{2(126)^2}\right] \cdot 36 \cdot (4.18) \cdot (.99) = 122 \text{ k}
\]

\[
P = 86 \text{ k} < Pa
\]

3B. Check local buckling. Element exceeds width-thickness ratio of applicable noncompact value, therefore it is classified as slender and shall be subject to a reduction factor Qs.

\[
b/t \leq 76/Fy^{0.5} = 76/(36)^{0.5} = 12.7
\]

\[
\frac{b}{t} = \frac{4}{\sqrt{16}} = 12.8
\]

\[
Qs = 1.34 - .00447(b/t)(Fy)^{0.5} = 1.34 - .00447(12.8)(36)^{0.5} = .99
\]
Kapue Bridge
Pier 5-6

Member end connections (beam elements)

~Critical rigid frame member end forces is due to beam element 125 (transverse direction)

\[ P = 86.6 \text{ K} \]
\[ M = 5.5 \text{ K'} \]
\[ P/2 = 43.3 \text{ K} \]
\[ M/2 = 33 \text{ K'} \]

\[ \sqrt{14.4^2 + 5.5^2} = 15.4 \text{ K} < 23 \text{ K} \]
Kapue Bridge
Pier 5-6

Connection at tower base: (2) 1-5/8" diameter anchor bolts

~Kapue pier 5 transverse direction

\[ R_{UT} = 139 \text{ K} \]
\[ R_{UV} = 69 \text{ K} \]
\[ R_{NT} = 186 \text{ K} \]
\[ R_{NV} = 116 \text{ K} \]

\[ \left( \frac{139}{186} \right)^2 + \left( \frac{69}{116} \right)^2 = |q| < 1.0 \]
Kapue Bridge
Piers 1-7

Column Base Plates:

Maximum compressive force = 360 K

Required area of base plate = 360/(.85f'_c)

f'_c = 4 ksi

A_{req} = 106 \text{ in}^2

Plan dimensions of base plate = 34" x 30"

Area = 1020 \text{ in}^2 > 106 \text{ in}^2

Plate thickness according to Cantilever method:

\[ tp = 1.41(n)(fp/Fy)^5 \]

\[ n = \text{distance from profile dimensions of column to edge of base plate} \]

\[ fp = P/BN \]

B = width of base plate

N = length (depth) of base plate

P = column load

\[ Fy = \text{yield stress of steel plate} \]

\[ tp = 1.41(4.5)(.423/36)^5 = .69" \]

actual base plate thickness = .75" > .69"

Foundation bearing:

Concrete pedestal footings are embedded in solid rock. The footings are prevented from sliding, uplifting, overturning and are adequate in bearing.
rocking analysis
Kapue Pier 7
Transverse direction

Determine P load for overturning about point A

\[ P = 129(17.9) + 129(11.4) + 129(2.4) + 45(29.2) + 45(20.2) + 6(14.65) \]
\[ P = 32.25 \]

\[ P = 280 \text{ k} \]
Kapue Bridge
Pier 7

Diagonal elements (tension members)

1A. Member stresses are less than or equal to 36 ksi for all tension elements in the transverse and longitudinal direction by inspection of SCPUSH-3D member forces.

1B. Member strains are less than 0.15 for all tension elements in the transverse and longitudinal direction by inspection of SCPUSH-3D member forces.
Kapue Bridge
Pier 7

Column elements (compression members)

2A. Check stability interaction criterion.

\[
P/P_a + C_m(1-P/P_e)M/M_y \leq 1.0
\]

\[
\frac{222}{624} + \frac{0.22}{(1-\frac{222}{624})} \cdot \frac{4.6(12)}{19.44}
\]

\[
= 0.36
\]

2B. Check yielding interaction criterion.

\[
P/P_y + M/M_y \leq 1.0
\]

\[
\frac{222}{635} + \frac{4.6(12)}{19.44} = 0.38
\]

2C. Check local buckling.

Built-up column section will not local buckle.

2D. Check overall buckling.

\[
P/P_a \leq 1.0
\]

\[
\frac{222}{624} = 0.36
\]
Kapue Bridge
Pier 7

Column elements (compression members)

~Critical rigid frame element is element 10 (transverse direction)

\[ P = 222 \text{ K} \]
\[ M_1 = -4.4 \text{ K} \cdot \text{ft} \]
\[ M_2 = 4.6 \text{ K} \cdot \text{ft} \]
\[ L = 14.4 \text{ ft} \]
\[ A = 17.64 \text{ in}^2 \]
\[ r_x = 5.04'' \]
\[ r_y = 5.04'' \]

\[ K_{xL}/r_x = 0.7(14.4)(12)/5.04 \approx 24.0 \]
\[ K_{yL}/r_y = 0.7(14.4)(12)/5.04 \approx 24.0 \]

\[ P_{Ax} = (1 - (K_{xL}/r_x)^2/(2(Cc)^2))F_yA = (1 - (24.0)^2/(2(126)^2))36(17.64) \approx 624 \text{ K} \]

\[ P_{Ay} = (1-(K_{yL}/r_y)^2/(2(Cc)^2))F_yA = (1 - (24.0)^2/(2(126)^2))36(17.64) \approx 624 \text{ K} \]

\[ P_e = \pi^2EI/(K_{xL})^2 = 6338 \text{ K} \]

\[ P_y = F_yA = 36(17.64) = 635 \text{ K} \]

\[ M_y = S_x \min F_y = 54(36) = 1944 \text{ K} '' \]

\[ C_m = 0.6 - 4(M_1/M_2) = 0.22 \]

.86
Kapue bridge
Pier 7

Concrete column pedestal design:

Two computer models were used to simulate inelastic behavior due to seismic activity.

Model #1: The connection between the steel frame above and the concrete columns below were modeled as a pin connection. The concrete pedestals (columns) were fixed at the base 18' below the steel and concrete connection.

Model #2: The connection between the steel frame above and the concrete columns below were modeled as a pin connection. The existing concrete pedestals (middle two columns) were fixed at the base 18' below the steel and concrete connection and the new concrete pedestals (outer two columns) were modeled as roller supports.

The maximum displacement of the concrete columns for both models are very small due to its relative stiffness to the steel built-up columns above. The concrete pedestals are 3.5' x 4.5' in cross section and remain in the elastic range when subject to dynamic loads.

An interaction diagram of axial force versus moment capacity is provided for reference. The maximum axial load and moment on the concrete columns at the base are 267 kips and 971 kip-ft respectively.
f'_c = 4.0 ksi
f_y = 40.0 ksi
8#9 0.4%
A_{st} = 8.00 in^2
Other cc = 3.37 in
spacing = 15.93 in
\text{x} = 551124 in^4
\text{x} = 0.00 in
I_y = 333396 in^4
\text{y} = 0.00 in

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Licensed To: Licensee name not yet specified.

Project:
Column Id:
Engineer:
Date: 02/19/99  Time: 11:35:38
Code: ACI 318-89
Version: 2.11

File name: UNTITLED.COL
Material Properties:
E_c = 3834 ksi  \quad \varepsilon_u = 0.003 \text{ in/in}
f_c = 1.20 ksi  \quad E_s = 29000 ksi
\beta_1 = 0.85
Stress Profile: Block
Reduction: \phi_c = 1.00  \quad \phi_b = 1.00

Slenderness not considered x-axis
kapiolani bridge
Pier 7

Beam elements (compression members)

~ Critical member element is element 4 (transverse direction).

3A. Determine allowable axial force (slender element).

\[ P_a = (1 - (K_l/r)^2/(2C_c^2))F_yA(Q_s) \]

\[ K_l/r = \frac{(1.9)(9.12)}{2.1} = 51.4 \]

\[ C_c = (2\pi^2E/F_y)^{0.5} = 126 \]

\[ F_y = 36 \text{ ksi} \]

\[ A = 5.72 \text{ in}^2 \]

\[ Q_s = 98 \]

\[ P_a = \left[ 1 - \frac{(51.4)^2}{2(126)^2} \right] 36(5.72)(98) = 185 \text{ k} \]

\[ P = 30 \text{ k} < P_a \]

3B. Check local buckling. Element exceeds width-thickness ratio of applicable noncompact value, therefore it is classified as slender and shall be subject to a reduction factor \( Q_s \).

\[ b/t \leq 76/F_y^{0.5} = 76/(36)^{0.5} = 12.7 \]

\[ \frac{b}{t} = \frac{5}{.7} = 7.3 \]

\[ Q_s = 1.34 - .00447(b/t)(F_y)^{0.5} = 1.34 - .00447(7.3)(36)^{0.5} = 98 \]
Kapae Bridge

Pier 7

Member end connections (beam elements)

~Critical rigid frame member end forces is due to beam element 4 (transverse direction)

\[ P = 30.3 \, K \]

\[ M = 5.2 \, K' \]

\[ P/2 = 15.2 \, K \]

\[ M/2 = 31.2 \, K'' \]

\[ \nabla_1 = \sqrt{(5.1)^2 + (5.2)^2} = 7.3 \, K < 23 \, K \]
Kapua Bridge

Pier 7

Connection at tower base: (2) 1-5/8" diameter anchor bolts

~Kapua pier 7 transverse direction

\[ R_{ut} = -219 \text{ K (compression)} \]

\[ R_{uv} = 54 \text{ K} \]

\[ R_{ut} = 186 \text{ K} \]

\[ R_{uv} = 116 \text{ K} \]

\[ 54 \text{ K} < 116 \text{ K} \]

\[ 54 \times .47 < 1.0 \]
Kapue Bridge
GTSTRUDL elastic analysis
2-dimensional model
"locked up" analysis
tension members only
longitudinal direction

\[ \text{Force} = 750 \text{ k} \]
\[ K_1 = 156 \text{ k/ft} \]
\[ \text{displacement} = \frac{F}{K} = \frac{750}{156} = 4.81" \]

\[ w = \sqrt{\frac{k}{m}} = \sqrt{\frac{156(386)}{813}} = 8.61\frac{1}{s} \]

\[ T = \frac{2\pi}{w} = \frac{2\pi}{8.61} = .73 \text{ s} \]

From normalized response spectra,

\[ T = .73 \text{ s} \quad \rightarrow \quad \text{spectral acc.} = 1.47 \text{ eff peak ground acc.} \]

\[ \text{spectral acc.} = 1.47(.42 \text{ g}) = .62 \text{ g} \]

\[ \text{Force new} = \text{mass new} \text{ (g)} \]

\[ = \frac{812}{g} \cdot (.62 \text{ g}) = 504 \text{ k} \]

\[ \text{Displacement new} = 4.81 \left(\frac{504}{156}\right) = 3.23" \]
Kapue bridge
GTSTRUDL elastic analysis
3-dimensional model
"locked up" analysis
tension members only
longitudinal direction

Force = 3000 k

\[ K_T = \frac{633}{K_{IN}} \]

\[ \text{displacement} = \frac{F}{K} = \frac{3000}{633} = 4.74" \]

\[ w = \sqrt{\frac{K}{m}} = \sqrt{\frac{633(366)}{3264}} = 8.65' \]

\[ T = \frac{2\pi}{W} = \frac{2\pi}{8.65} = .73 s \]

from Normalized response spectra,

\[ T = .73 s \quad \frac{\text{spectral acc.}}{\text{eff peak ground acc.}} = 1.47 \]

spectral acc. = 1.47(.42g) = .62g

\[ \text{Force}_{new} = \frac{\text{mass}_{new}(g)}{g} = \frac{3264}{g} (.62g) = 2024 k \]

\[ \text{Displacement}_{new} = 4.74 \left( \frac{2024}{3000} \right) = 3.20" \]