SEISMIC PERFORMANCE ASSESSMENT OF CONCRETE BRIDGES DESIGNED BY DISPLACEMENT-BASED METHODS

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Abstract

With the advent of performance-based design, it is necessary to consider the performance of bridges as an intrinsic part of the design process. However, even when performance is measured in terms of deformations and displacement-based design is utilized, it is of interest to know whether designs actually result in the desired performance under ground shaking representative of the design hazard. Four case studies are designed in this paper, ranging from an elasto-plastic oscillator to a three-span continuous prestressed concrete bridge. The distribution of peak responses was assessed for each case study in reference to the original target displacement used for design.

Introduction

Performance-based design (PBD) aims to improve performance by defining performance criteria that must be satisfied at more than one earthquake level. Generally, better performance is expected for important structures and smaller earthquakes, while lower levels are required for ordinarily structures and more rare events. Performance is no longer related to collapse prevention or life safety only; deformations, functionality, economic losses, and downtime are additional criteria (Mackie and Stojadinovic, 2007; Mackie et al., 2010). Yet, there is a question as to whether the PBD procedure actually results in a structure that meets the performance objectives.

PBD explicitly considers how a structure is likely to perform. The performance assessment requires detailed analysis because it becomes an intrinsic part of the design process. A good preliminary design will reduce or eliminate the need for iteration required to meet the performance objectives. Design procedures that are useful within this framework must: (i) Take any combination of earthquake level and performance criteria; (ii) Produce a design that meets the target performance; (iii) Be rational and easy to execute. Both force-based and displacement-based procedures are potential design methodologies for applicability as PBD tools; however, each with differing merits in terms of the three criteria listed above.

Regardless of the design methodology adopted; however, design should be carried out considering the following. First, seismic resistant bridges should have a

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simple configuration, such that their behavior can be easily modeled and analyzed. The chosen configuration should also aim to include energy dissipation in different components of the structure with ductile mechanisms. Second, in conventional bridges, pier columns provide the primary energy dissipation mechanism, while abutments can provide additional energy dissipation (Priestley et al., 1993). Recommended earthquake resisting systems for bridges are given in the Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2009). Finally, capacity design principles must be applied in all cases to protect the components outside the ductile mechanism and to prevent non-ductile modes such as shear.

Force-Based Design (FBD) and Displacement-Based Design (DBD)

Seismic design of bridges can be accomplished following different approaches. The traditional procedure is force based since damage in the structure is controlled by the assignment of a certain level of strength. The procedure uses strength reduction factors to reduce the elastic force demand while considering importance, assumed ductility capacity, over-strength and redundancy in the structure. FBD is found in the AASHTO LRFD Bridge Design Specification (AASHTO, 2004) and was first adopted by AASHTO in 1983 following recommendations of the Applied Technology Council (ATC, 1981).

There are several problems attributed to FBD. First, strength is used as a means to control damage, although these parameters do not correlate well. Second, it is assumed that strength and stiffness are independent. Third, force reduction factors (R) are used assuming that the ductility demand will be the same for each type of structure. Finally, the R factors are given generally for a single level "no-collapse" design. Multi-level design would require the specification of different R values.

After the Loma Prieta earthquake in 1989, extensive research has been conducted to develop improved seismic design criteria for bridges, emphasizing the use of displacements rather than forces as a measure of earthquake demand and damage in the structure. Research has also focused on the application of capacity design principles to assure ductile mechanisms and concentration of damage in specified regions. Several DBD methodologies have been developed including:

- Direct Displacement Based Design (DDBD) (Priestley, 1993)
- MCEER/ATC-49 Recommended LRFD guidelines for seismic design of bridges (ATC, 2003)
- Seismic Design Criteria (SDC) of Caltrans (Caltrans, 2004)
- Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2009)

DBD has gained popularity in the last fifteen years, as it addresses several shortcomings of the conventional FBD procedure, while serving as a useful tool for performance-based seismic design. The primary difference between DBD and FBD is that the former uses displacement as a measure of seismic demand and also as an indicator of damage in the structure. DBD takes advantage of the fact that displacement correlates better with damage than force. DBD also overcomes serious problems of FBD such as ignoring the proportionality between strength and stiffness and the generalization of ductility capacity through the use of force reduction factors. DBD can be used with any combination of earthquake level and performance criteria.

Both conventional DBD and direct DBD are compared briefly below; however, DDBD is selected as the design procedure for the case studies presented in this paper. The primary differences in the AASHTO (2009) LRFD procedure and DDBD are linearization and the execution. The displacement demand assessment procedure in the AASHTO LRFD Seismic guide uses elastic analysis and the equal displacement approximation (Veletsos and Newmark, 1960) to obtain inelastic displacement demands (an amplification factor is used with short period structures). In the elastic analysis the structure is modeled with cracked section stiffness. DDBD uses the equivalent linearization to overcome the limitations of the equal displacement approximation (Suarez, 2008). In execution, the AASHTO LRFD uses a demand/capacity assessment procedure. In contrast to this, DDBD goes directly from target performance to required strength. The amount of reinforcement does not need to be assumed during design.

Conventional Displacement-Based Design approach

The Seismic Design Criteria by Caltrans (2004) shifted towards displacement-based design in 1999 by consolidating ATC-32 recommendations (ATC, 1996). Currently, Caltrans has an iterative design procedure in which the lateral strength of the system (size and reinforcement of the substructure sections) is assumed at the beginning of the process. Then, by means of displacement demand analysis and displacement capacity verification, it is confirmed that the bridge has an acceptable performance, otherwise, the strength is revised and the process repeated.

In the demand analysis, the peak inelastic displacement demands are estimated from a linear elastic response spectrum analysis of the bridge, with cracked (secant to yield point) component stiffness. Then, elastic peak displacements are converted to peak inelastic displacements using an equal displacement approximation (Veletsos and Newmark, 1960) with modification for short period structures. Once the displacement demands are estimated, the procedure requires the verification of the displacement capacity of each pier by means of a pushover analysis. Finally, the substructure sections and protected elements are designed and detailed according to capacity design principles.

The AASHTO Guide Specifications for LRFD Seismic Bridge Design (2009) recognizes the variability of seismic hazard over the US territory and it specifies different Seismic Design Categories (SDC). Each SDC links seismic hazard to expected performance. The design procedure in the AASHTO DBD approach is in concept similar to the Caltrans approach. Depending on the configuration of the bridge, the demand analysis is performed by the uniform load method for regular bridges, while spectral modal analysis can be used for all bridges. The capacity verification can be done using implicit equations for seismic design category B or by pushover analysis for categories C and D. As with the Caltrans approach, and with the exception of seismic category A, the proposed guide requires the use of capacity design principles for the detailing of the substructure sections and protected elements.

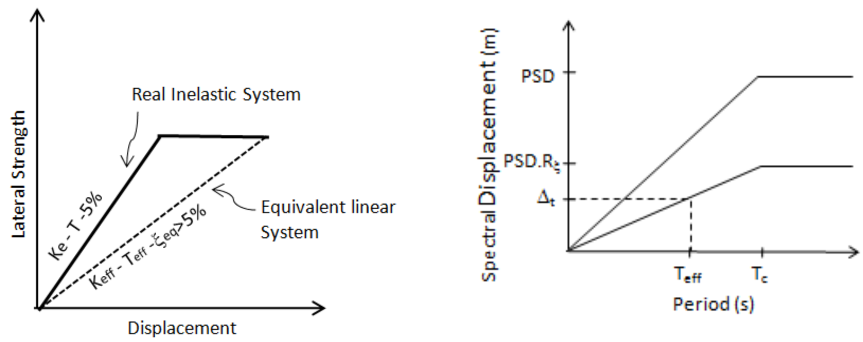
Current Caltrans and AASHTO approaches utilize acceleration spectrum curves to determine the displacement demand at the system level. The main limitations of this approach are:

- The use of the equal displacement approximation. Research conducted on displacement modification factors (FEMA 440, 2005) has shown that the ratio between inelastic and elastic displacement depends on period, hysteresis shape, and other factors. In addition to this, assuming that the elastic displacement demand equals the inelastic demand is not appropriate when additional damping exists in the structure as a result of soil-structure interaction or other sources.
- The use of acceleration response spectrum to compute displacement demand. The displacement spectrum seems a more rational source of seismic hazard for DBD.
- The procedure is iterative in nature since reinforcement in the pier sections must be guessed at the beginning of design. If the displacement capacity is ultimately less than the displacement demand, the process must be repeated increasing the amount of reinforcement. If the inverse occurs, no iteration is needed; however, the resulting design will be overly conservative.

Direct Displacement Based Design (DDBD)

DDBD has been conceived as a tool to achieve deterministic PBD, as a simple methodology that can be used to go from basic geometry to properly detailed sections and structural components. Research conducted in the last fifteen years have shown the method produces satisfactory bridge designs (Kowalsky et al., 1995, Calvi and Kingsley, 1995, Dwairi et al., 2006, Suarez and Kowalsky, 2007, Priestley et al., 2007), however a formal reliability study has not been yet conducted and the use of DDBD within the scope of probabilistic PBD requires further research. The DDBD method was initially proposed by Priestley (1993). In its current state, DDBD works with any combination of seismic hazard and performance criteria, and it is intended to produces structures that meet (theoretically in the mean), rather than be bounded by, the target performance. This makes DDBD a very attractive alternative for preliminary design since it reduces, and in some case might eliminate, the need for iteration in a general PBD procedure.

DDBD differs from the DBD procedure in the AASHTO Guide Specifications for LRFD Seismic Bridge Design in the use of an equivalent linearization approach and in the execution of the procedure. DDBD starts with the definition of a performance-based target displacement for the structure and returns strength required to meet the target displacement under the specified earthquake. The method is referred to as “direct” since, in contrast to the traditional DBD procedure of AASHTO or Caltrans, the reinforcement and thus the strength of the structure does not need to be assumed at the beginning of the design and modified iteratively until a demand/capacity check is satisfied.



(a) Equivalent Linearization (b) Displacement Spectrum

FIGURE 1 - Fundamentals of DDBD

DDBD uses an equivalent linearization approach (Shibata and Sozen, 1976) where an inelastic system at maximum response is modeled by an equivalent elastic system with secant stiffness (K_{eff}) and equivalent viscous damping (ξ_{eq}) (FIGURE 1a). A design objective must be defined as a combination of a performance criterion and design earthquake. The performance can be specified in terms of material strains, curvature, drift, or ductility in the piers. In all cases, consideration of abutment displacements as a limit state, as well as P-Δ effects of bridge piers should also be addressed. The design objectives can also be those stated for each SDC in the AASHTO LRFD Seismic Guide Specification. The design earthquake is represented by a displacement spectrum that is reduced to the level of damping of the structure (FIGURE 1b).

In most cases DDBD can be applied with simple hand calculations. Modal spectral analysis or pushover analysis are not required. A major limitation of this approach is the target displacement can only be estimated for simple pier configurations (the most common at least). Since a pushover analysis is not carried out, the flexibility of cap beams and rotation of foundations (for example) cannot be incorporated without some iteration. In addition, the procedure is direct (no iteration) only when the shape of the displacement profile is known. This scenario occurs only in bridges with regular distributions of mass and stiffness. Curved bridges are design as straight.

Analysis Procedure

A DDBD procedure is carried out for four bridge case studies in this paper. A location is selected in the central United States to define an equal hazard spectrum for all four case studies, except for the final bridge structure that has a higher design spectrum. After a target displacement is selected for each structure based on appropriate limit state definitions (specific to reinforced concrete bridges), the performance of the case study structures was assessed using nonlinear time history analysis (THA) with recorded ground motions. The probabilistic assessment allows confirmation of whether the target displacement is achieved in the mean, reflecting an unbiased design procedure. Due to the assumptions surrounding the DDBD procedure, meeting the performance objective in the mean is unlikely; therefore, this paper investigates whether the design procedure can be formulated to achieve a conservative design. Above and beyond this, it is demonstrated that under a given performance objective and probabilistic acceptance criterion, a modifier on the initial design displacement can be specified so that the procedure retains the advantages of being non-iterative (see FIGURE 2).

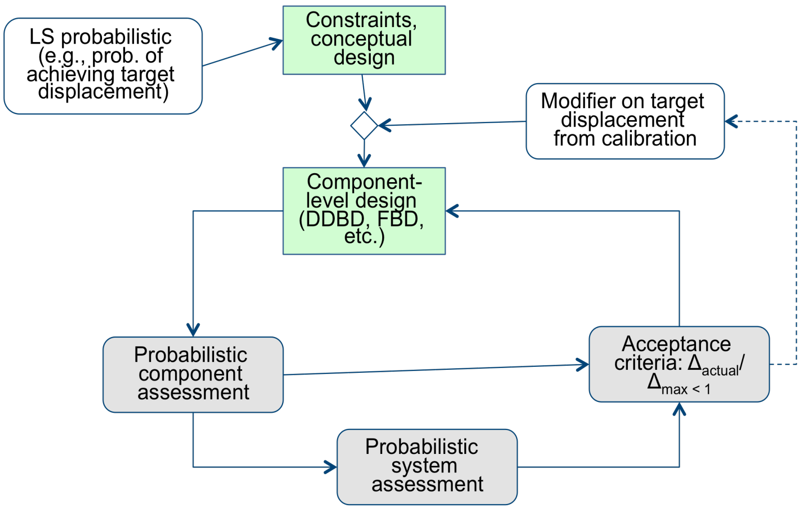


FIGURE 2 - Probabilistic considerations in DDBD and assessment

A complicating factor for nonlinear assessment of structures designed according to DDBD is that bridges are inherently three-dimensional (3D) systems. Two important phenomena are illustrated in the case studies presented in this paper: 1.) 3D excitation and response impact performance in ways that are difficult to account for in only longitudinal or transverse simplifications (or combination rules), and 2.) the bridge responds as a system, with contributions from several load-resisting components such as shear keys, abutments, foundations, and the superstructure itself. Therefore, assessment often yields different response quantities than the initial design. The choice of assessment procedure and degree of model complexity also influence agreement between design and assessment. Allowance for different assessment techniques (linear dynamic, nonlinear static, nonlinear dynamic, etc.) should be considered when proposing any modification factors on the initial target displacement; however, only nonlinear THA is considered in this paper.

Hazard, DDBD, and Case Study Details

A site near the New Madrid seismic zone was selected (-90.196, 35.212). Multi-level hazards are defined based on USGS seismic hazard maps. Spectral acceleration curves were generated for three hazard levels: 10%-, 5%-, and 2%-probability of exceedance in 50 years and 5% equivalent viscous damping. The acceleration spectra were converted to displacement spectra and then linearized (relationship between T and Sd). The peak spectral displacement was assumed to occur at the corner period of 3 sec. Based on extrapolation of the Sd curve, the Sd at corner period are 5.76, 16.1, 33.4 (cm) for each of the three hazard levels. The spectral displacement hazard at the 2% probability of exceedance in 50 year level was raised to 72 cm for the bridge in case study as it was originally designed for a site in California with substantially higher hazard.

Once the design objective has been selected, the main steps of the DDBD procedure are: (i) Select the dimensions of the components of the earthquake resisting system on the basis of past experience. (ii) Compute a target displacement based on the performance level for the structure. Depending on whether this is specified as a strain, ductility, etc., it may be necessary to use a plastic hinge model to relate these to peak displacements. (iii) Evaluate the effective mass and equivalent viscous damping for the system. Priestley et al. (2007) defined a ductility vs equivalent damping relationship. (iv) Compute the spectral reduction factor that corresponds to the equivalent damping level in the structure and find a reduced design spectrum. The spectral reduction factor for inelastic structures is based on Eurocode (1998). (v) Determine the required effective period, secant stiffness and required strength. (vi) Distribute the required strength, design plastic hinges and protected elements using capacity design principles.

The four case studies considered in this paper are: 1.) a single-degree-of-freedom elasto-plastic oscillator, 2.) a two-dimensional reinforced concrete bridge bent with a single column subject to transverse excitation, 3.) the same two-dimensional bridge bent subject to both lateral and vertical excitation, and 4.) a 3D reinforced concrete bridge with 3 continuous spans and explicit foundation and abutment representations.

Case Study 1: Single-degree-of-freedom (SDOF) system

The SDOF system was selected as the simplest case of design where the period of the structure (both loading and unloading) is constant and the yield point defines the perfectly plastic plateau. Therefore, the only unknowns are the period and the yield strength. A factor of 55 was used in the expression for equivalent damping of the elasto-plastic system. The benefit of using a SDOF oscillator is that it is not necessary to use nonlinear THA to assess the performance of the system, other approximate techniques can be readily used (such as R-μ-T relationships). The oscillator was assumed to have a yield displacement of 0.05 m (typically the yield displacement is obtainable from the structure's geometry) and a target ductility of 4. For the 2%-in-50-year hazard defined previously, the effective period becomes 3.28 sec and the target strength is 0.73 kN.

The properties of the oscillator were then used in two separate analyses. The first was to use a common relationship between R-μ-T to obtain the achieved ductility at the target spectral displacement. The second was to perform inelastic THA with the specified oscillator properties. A total of 160 ground motions were scaled to the target spectral displacement before performing the analysis and the distribution of achieved maximum displacements is shown in FIGURE 3 below. The individual ground motion realizations are shown in the top pane of the figure while the cumulative distribution function (CDF) is shown in the bottom pane. The actual time history CDF is shown with lower (LCB) and upper (UCB) bound indicators. Finally, the data is assumed to follow a lognormal distribution and two parameters are estimated using maximum likelihood.

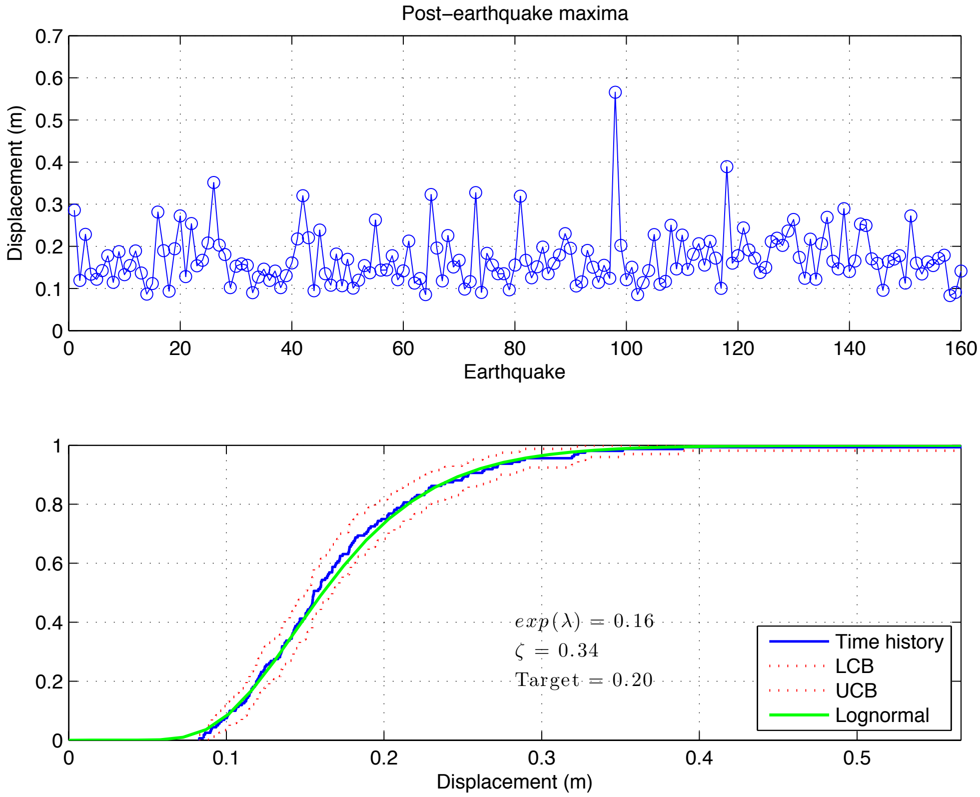


FIGURE 3 - Response statistics for SDOF oscillator in Case study 1

The mean displacement from the R-μ-T and THA analysis methods were 0.17 and 0.17 m, respectively (the THA mean was obtained from the method of moments considering the fitted lognormal parameters). An equivalent statement is that a system with 15% less strength (than that required by DDBD) would be required to exactly produce the target displacement. The R-μ-T relations provide only mean or central value information, but the THA provides the actual distribution of responses. It can be observed that the probability of exceeding the target design displacement is 0.26.

Case Study 2: Single bent with transverse excitation

A regular two-lane RC box girder bridge with single column piers that are integral with the superstructure is selected for a more realistic case study (FIGURE 4). The piers are supported on a rigid pile group. The spans are 50 m long and the weight of the superstructure is 180 kN/m. The bridge is assumed to behave like a SDOF system, but exhibits material and geometric nonlinearity typical of a reinforced concrete structure rather than an idealized bilinear elasto-plastic material. The height is 6 m (pier) + 0.8 m (rigid within superstructure to center of mass), and the column diameter is 1.3 m. Other values that were assumed for the analysis and design are: expected concrete compression strength (f'_{ce}) 45000 kPa, expected yield strength of main reinforcement (f_y) 462000 kPa, expected yield strength of transverse reinforcement (f_{ys}) 462000 kPa, diameter of longitudinal bars (d_b) 32 mm, diameter of spiral (d_{bs}) 22 mm, pitch of spiral (s) 100 mm, and 50 mm cover to main reinforcement. The design assumptions result in a volumetric spiral ratio (ρ_s) of 0.013 and an axial load ratio (ALR) of 0.154.

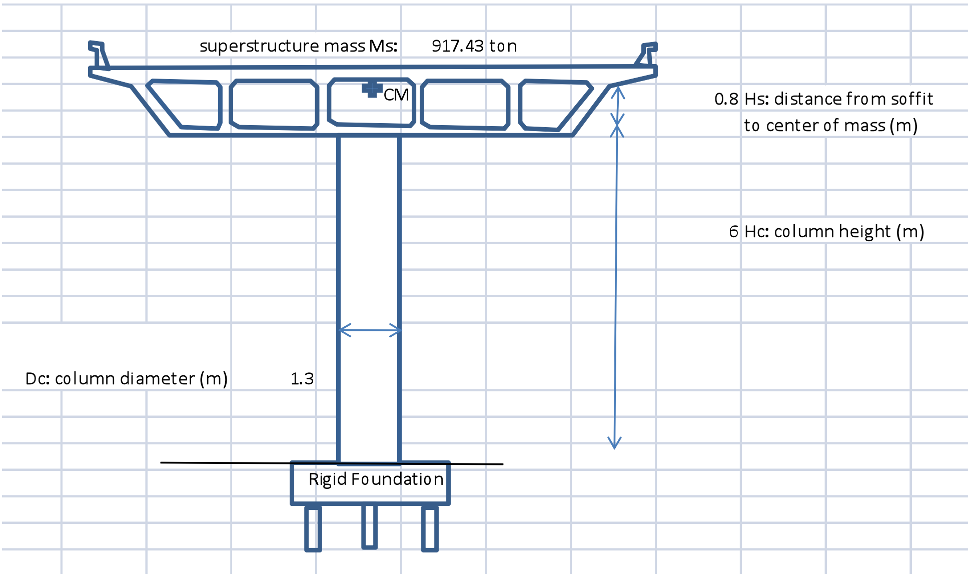


FIGURE 4 - Design schematic for single-column bent transverse analysis case study

The target displacement was defined by the drift ratio required for initiation of spalling, as defined by Berry and Eberhard (2003), or 0.13 m. Nonlinear THA was performed on the bent using 160 ground motions scaled to the spectral displacement demand at the initial elastic period. The initial elastic period was calculated after gravity load analysis but without any equivalent linearization or secant approximations. As with the SDOF oscillator, the distribution of maximum displacements were obtained and plotted in FIGURE 5. The mean achieved displacement is 0.11 m and the probability of exceeding the target displacement is 0.27.

Case Study 3: Single bent with transverse and vertical excitation

The same reinforced concrete bent from the previous case study is reused, but an additional two components of excitation were added. The case study demonstrates the effects of varying axial loads and potential P-Δ effects without the need for combination rules; however, is only a simple extension as it does not consider rotational inertia or boundary conditions at the top of the column. For ground motion amplitude, the design spectrum was treated as the geometric mean of the two lateral components. The response metric was taken as the maximum of the instantaneous vector combination of the two orthogonal lateral components (known as the square-root-sum-of-squares). While (as expected for circular columns) the response is very similar to the previous case study, the mean achieved displacement increased slightly to 0.14 m (FIGURE 6). This increase brings to the probability of exceeding the target displacement to 0.50, but is more representative of the difference in the lognormal fit to the data than the actual change in the mean. The change in shape in the distribution is indicative of several larger displacement realizations obtained from bidirectional displacement orbits.

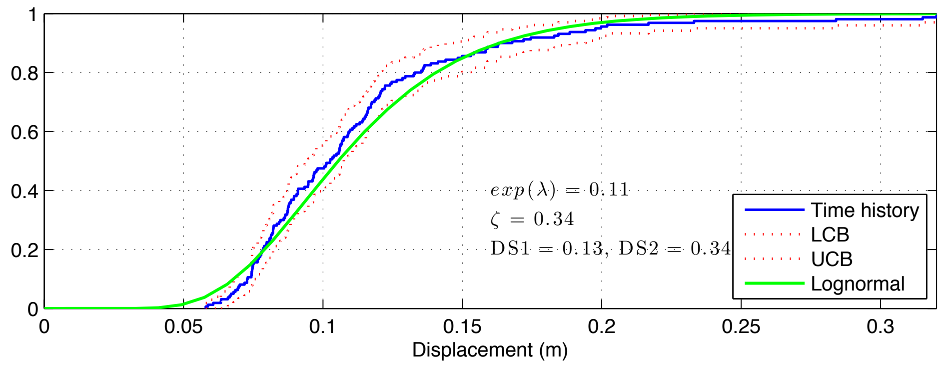


FIGURE 5 - Distribution of maximum transverse displacements for Case study 2

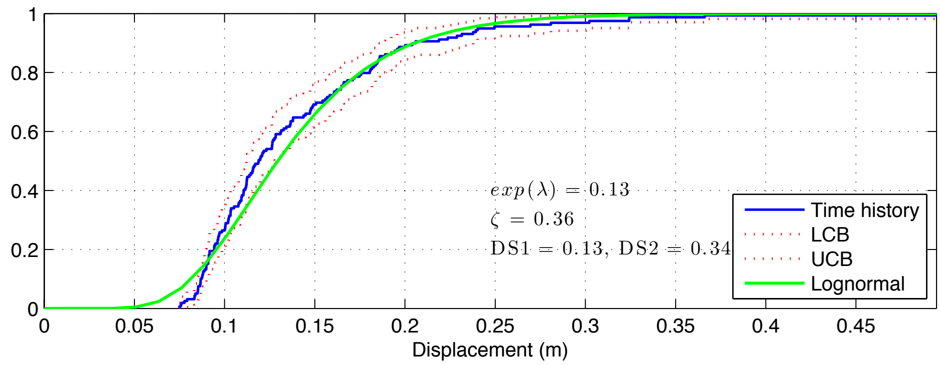


FIGURE 6 - Distribution of maximum SRSS displacements for Case study 3

Case Study 4: Typical three-span California bridge

The problem is generalized to a 3D case (both structure and excitation) to assess the effect of multiple components (lateral) of excitation and response, as well as the impact of system performance on components design according to the DDBD procedure. Explicit representations of the stiffness and strength of the abutments and superstructure are included in the assessment model. The case study is taken directly from the LRFD design example (AASHTO, 2006) for a bridge typical to California that falls into SDC D. Conventional DBD was performed for this bridge in the example and subsequently DDBD was also performed on the same structure (Suarez and Kowalsky, 2010). The DDBD detailing to achieve a target displacement of 0.64 m is used for assessment in this case study.

The bridge has three spans of 38.41, 51.21, and 35.98 m with a continuous prestressed reinforced concrete box girder superstructure, as shown in FIGURE 7.

The two bents are skewed 20 degrees and have two 1.83 m diameter columns supported on piles (FIGURE 8). Column height varies from 13.4 m at bent two to 14.3 m at bent three. The columns are pinned at the bottom and fixed to an integral bent in the superstructure. The bridge is founded on seat type abutments with elastomeric bearings and a break-off wall once the gap closes in the longitudinal direction. Exterior shear keys prevent transverse motion under lower intensity motions and service loads. The superstructure is capacity designed to remain elastic at the target displacement and is therefore modeled using an elastic section with cracked properties. No explicit representation of the tendons or mild steel was created in the analytical model.

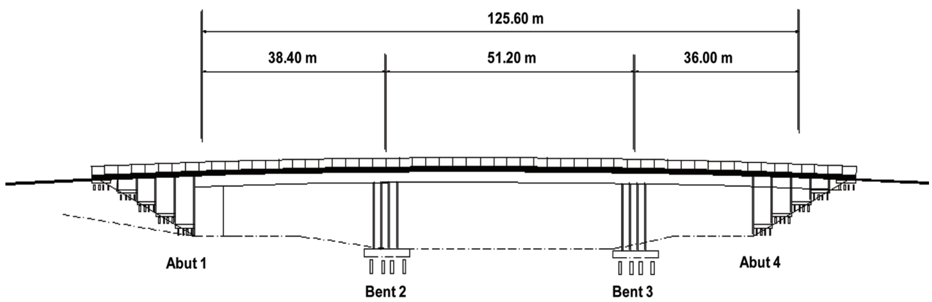


FIGURE 7 - Elevation of 3-span continuous case study bridge (Suarez and Kowalsky, 2010)

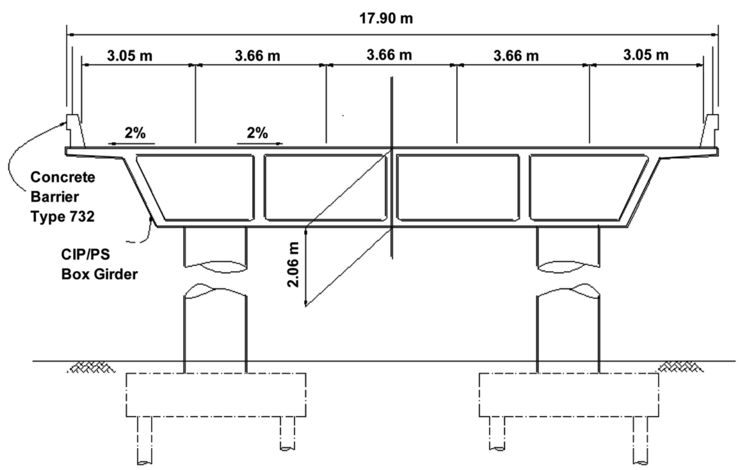


FIGURE 8 - Bent configuration and superstructure cross section for 3-span continuous case study bridge (Suarez and Kowalsky, 2010)

The expected concrete compression strength was 36000 kPa, expected yield strength of main reinforcement 455000 kPa, diameter of spiral 25 mm, pitch of spiral 125 mm, and 50 mm cover to main reinforcement were consistent with the DDBD design. The superstructure elastic properties were obtained from the LRFD design example appendix and were factored by 0.5 for cracked moment of inertia and 0.25 for cracked torsional constant. The bent cap is modeled explicitly and also contains cracked elastic properties based on initial gross dimensions. The two columns per bent are modeled with a rigid extension into the bent cap where the elements for the superstructure are placed at the center of gravity. An integral diaphragm at the abutments allows for the placement of abutment spring elements at the transverse extremes of the superstructure cross section. To be consistent with the design assumptions, both the longitudinal and transverse abutment responses were assumed elasto-plastic. The longitudinal response is mobilized only in compression (movement of the deck into the backwall). Due to the skew of the bridge, the abutment springs were aligned parallel and perpendicular to the abutment diaphragm (not in the global bridge longitudinal direction).

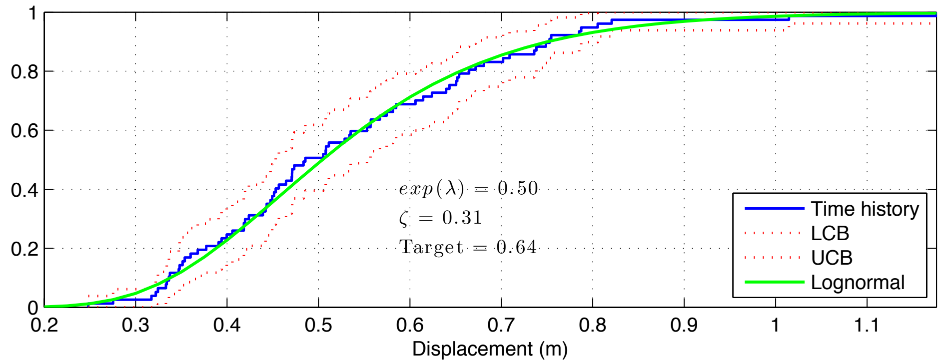


FIGURE 9 - Three-span bridge maximum SRSS displacements for Case study 4

A total of 80 ground motions were used for nonlinear THA. The CDF of maximum response is shown in FIGURE 9. The mean response was 0.53 m and the probability of exceeding the target displacement was 0.22. Similar to the previous case studies, the DDBD target displacement is conservative to the maximum response achieved using nonlinear THA. However, in the previous case studies it was possible to control for many variables that potentially differ between DDBD and analysis. This more realistic case study contains complete 3D nonlinear element interaction, 3D excitation, nonlinear geometry, and 3D response. In addition, the bents are skewed, causing interaction between the longitudinal and transverse bridge directions. Assumptions were also made on the analytical side, such as not explicitly representing the foundations or the fill above the pile caps, the tendons and axial forces in the superstructure, the mild steel and concrete nonlinearity in the box girder and bent cap, selection of equivalent viscous damping and damping model, period selected to decide scale factor for each ground motion, and shear deformations and any other non axial-flexural modes.

Design for Target Performance Objective

While this paper has yet to fully characterize the reliability of the DDBD procedure, it does provide some insight into the target vs achieved displacements for a variety of bridge structures. Based on the consistency of the results, it was postulated that the probability of exceeding the target displacement could be specified apriori as part of the performance objective for design. For example, for Case study 1, if the target displacement is 0.2 m and the maximum permissible probability of exceedance is 50% (most likely too high from a risk perspective), then the inverse problem can be solved. A modifier on the target displacement (0.28 m) will result in a new period and required strength (0.523 kN). Assessment of this modified system yields a mean displacement of 0.2 m. For the SDOF system, this phenomenon is easily explored analytically using the expression for spectral factor based on the equivalent damping. Continuing work will demonstrate the relationship between the distribution of this parameter and the resulting responses. The numerical level of conservatism (and therefore the ability to achieve target risk levels for each displacement) will be demonstrated and related to the brief inverse problem described in this section. It is also worth noting that as the target displacement (with modifier) is increased, the selection of recorded ground motions that meet the target spectral displacement for design diminish. Therefore, scale factors on ground motion amplitude are used that would likely cause a bias in the observed response distribution.

Conclusions

This paper investigates performance-based assessment of the direct displacement-based design (DDBD) procedure. Four case studies were selected ranging from an elasto-plastic oscillator to a three-span continuous prestressed concrete box girder bridge. Each case study was designed using DDBD for a specified hazard level. Subsequently, a nonlinear time history analysis was conducted to assess the performance of each, or more specifically, the probabilistic distribution of peak displacement responses in reference to the original target displacement used for design. It is demonstrated that, consistent with earlier findings, the DDBD leads to a slightly conservative design whereby the target displacement is exceeded less than 50% of the time. In all the case studies (except Case 3), the probability of exceeding the target displacement is approximately 25%. It was demonstrated that this information enables a modifier to the original target displacement to achieve a specified risk level (acceptable probability of exceeding target displacement) without iteration. Further work is necessary to determine the nature of the response distribution for different structure types and modeling assumptions before such a technique can be used more broadly to achieve performance-based design objectives.

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