20-15 LATERAL SPREADING ANALYSIS FOR NEW AND EXISTING BRIDGES

Introduction

This memo describes a procedure to estimate the deformation demands (and capacities) of bridge foundations and abutments resulting from liquefaction induced spreading ground (i.e. horizontal ground displacement). Designers wishing to learn more about liquefaction can refer to Caltrans *Geotechnical Manual*, *Memo to Designers* (MTD) 20-14, and *Seismic Design Criteria* (SDC) Sections 2.2.5 and 6.1.2.

Lateral spreading is caused by the accumulation of incremental displacements that develop within liquefied soil under cyclic loading. Depending on the number and amplitude of stress pulses, lateral spreading can produce displacements that range from a few inches to tens of feet. For the purposes of this memo, the definition of lateral spreading is extended to the case of flow liquefaction. Flow liquefaction occurs when a slope becomes unstable under static loading due to strength loss caused by liquefaction. Flow liquefaction is characterized by slumping behavior and generally large deformation. The procedures described in this memo apply to both cyclic deformation and slumping behavior.

Excessive load or displacement demands caused by lateral spreading can be addressed using ground improvement techniques or structural enhancement of the bridge. Either option can be expensive. Generally, structural options are preferred but ground improvement options should be considered as well. Conservatism that might be appropriate in other design situations may come at an unacceptably high cost when applied to lateral spreading evaluation. The following evaluation procedure seeks to provide a best estimate of lateral soil displacement and the resulting displacement demand on the bridge.

New bridges in potentially spreading soil must be supported on ductile and strong foundations. Ductile piles and shafts are allowed to form two plastic hinges with allowable ductility demands up to five ($\mu_D \leq 5$). The recommended performance criteria reflect that SDC compliant bridges have the capacity to withstand large deformation demands without collapse. The performance criteria for existing bridges are described in the "Capacity Estimation" section of this memo.

The following sections outline the design procedure for evaluating lateral spreading. These sections provide the Geotechnical Designer (GD) and the Bridge Designer (BD) with the basic steps required to estimate the displacement demand on bridge members resulting from lateral spreading. Attachment 1 provides recommendations for the development of *p-y* curves and the construction of an equivalent single pile model.

Demand Estimation

Overview of the Evaluation Procedure

The displacement demand evaluation procedure relies on an equivalent nonlinear static analysis methodology. A pseudo-static slope stability analysis is used in conjunction with a sliding block analysis to estimate horizontal displacement demand. A reduction in ground displacement resulting from the restraining action of the foundation is assessed by imposing a condition of displacement compatibility between the deflecting foundation and spreading soil. The loading of the foundation by the spreading ground is assessed using a Winkler spring foundation model where the base of the *p-y* springs is displaced an amount equal to the ground displacement, as shown in Figure 1(b). A soil displacement profile is imposed on a Winkler spring foundation model. The GD determines the amount of displacement from a sliding block type analysis where the peak displacement is determined by the yield coefficient of the block (k_y) and the peak ground acceleration (PGA). A slope stability analysis is performed to determine the dimensions of the failure mass and its corresponding yield coefficient. This yield coefficient is then used in the sliding block analysis. Generally, it is preferred that soil displacements are imposed on the foundation(s) within the context of a global bridge model so that deformations can be more accurately distributed throughout the bridge. For shorter bridges, where a global model does not offer substantial refinement, a single bent analysis is sufficient.

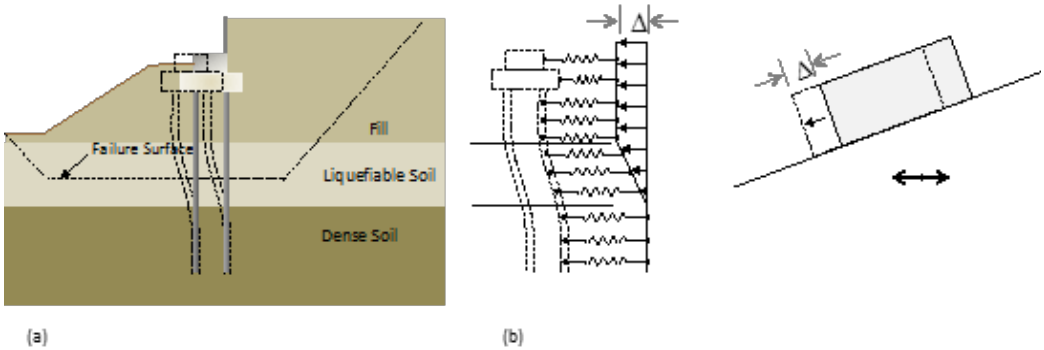


Figure 1 (a) Single bent idealization of the lateral spread-foundation interaction problem. (b) The analytical model relies on a soil displacement profile imposed on a Winkler spring foundation model. The displacement profile is estimated by a sliding block procedure that uses the yield coefficient determined from a slope stability analysis.

The evaluation procedure outlined above is performed for the case of liquefaction. Evaluation of the liquefaction potential at a bridge site and modification of soil strength to account for the effects of liquefaction are described in the "Liquefaction Trigerring and Residual Strength" section of this memo.

Liquefaction Triggering and Residual Strength

The lateral spreading analysis begins with an assessment of liquefaction triggering potential by the GD. The procedures for triggering analysis are provided in the 'Liquefaction Evaluation' section of the Geotechnical Manual. Soil strata that have a triggering factor of safety (FS_{liq}) less than 1.05 should be modeled as soft clay with strength equal to the liquefied soil's residual strength determined according to equation (1) (Kramer and Wang, 2015). In equation (1), $(N_1)_{60}$ is the correlated standard penetration resistance (in blows per foot), σ'_v is the effective stress (lbs/ft^2) and S_r is the soil's residual strength (lbs/ft^2).

$$S_r = 2.116 \cdot \exp\left\{-8.444 + 0.109(N_1)_{60} + 5.379\left(\frac{\sigma'_v}{2116}\right)^{0.1}\right\} \quad (1)$$

For strata with $1.05 < FS_{liq} < 1.20$, the frictional strength should be reduced by a factor of 0.6. This can be achieved by scaling the friction angle by 0.65.

Liquefaction Depth Limits

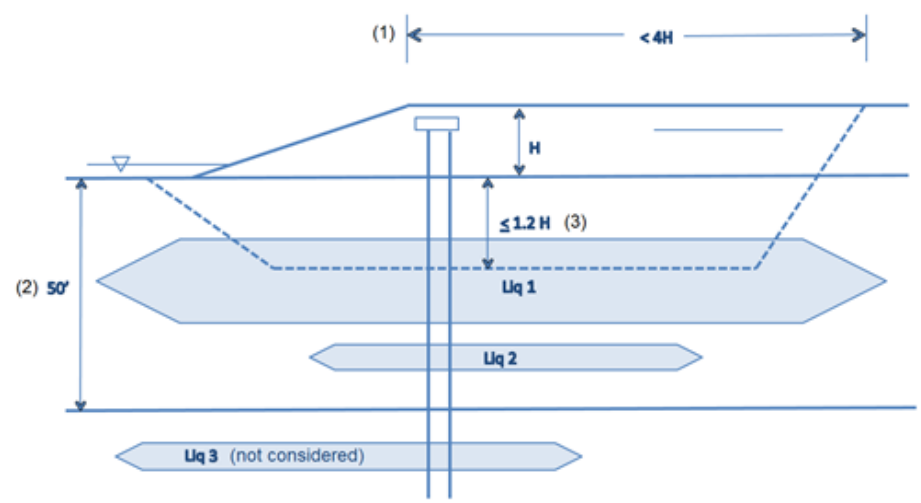
Generally, liquefaction is more likely to occur at shallower depths since shallow soil hasn't yet benefited from aging effects that tend to promote a more stable soil structure. Shallow soil will also be subject to larger dynamic shear strains than deeper soil. Deeper soil can still liquefy under strong shaking but the impact in terms of lateral ground displacement is mitigated by the restraining action of overlying soil. For these reasons, liquefiable strata greater than 50 feet deep, measured from the toe of the slope or the deepest portion of a cut channel, should not be considered in the lateral spreading analysis. Additional depth related criteria are provided in the "Sliding Block Analysis" section shown below.

Sliding Block Analysis

A pseudo-static slope stability analysis is performed to evaluate the stability of the slope under liquefied conditions. If the factor of safety (FS) < 1, flow liquefaction should be assumed. Although there are no procedures for estimating ground displacement under flow conditions, displacements can be assumed to be large (exceeding 10 feet). Since corresponding foundation loading will reach a maximum at a displacement demand less than 5 feet, a demand of 5 feet can be used for analysis.

Calculation of the Yield Coefficient

If the pseudo-static factor of safety (*FS*) > 1 under liquefied conditions, the yield coefficient k_y is determined by finding the horizontal acceleration required to achieve a *FS* = 1. Determination of k_y implicitly assumes that the entire slope is moving in concert. Not considered is the varying level of shaking that occurs with depth nor the spatial variability that results from varying soil conditions or topographic effects. An effect of this perfect coherence is that when horizontal acceleration is included in the slope stability analysis, it tends to drive critical failure surfaces to unrealistic proportions since the horizontal driving force scales linearly with the failure mass. To better reflect actual slope failure behavior, constraints must be imposed on the critical failure surface used to determine k_y. Generally, critical failure surfaces have a wedge type shape such as that shown in Figure 2. This figure also provides recommended limits on both the lateral and vertical extent of critical failure surfaces for k_y calculation. Normally, the failure surface should be at the depth with the lowest factor-of-safety for liquefaction triggering. In cases where soil properties are approximately uniform within the liquefiable stratum, the failure surface should be placed in the center of the stratum.



(1) Failure surfaces are constrained laterally to less than 4H as measured from the slopecrest.

(2) Any liquefiable stratum deeper than 50 feet, as measured from the lowest topographical feature, is not considered.

(3) In the case of multiple liquefaction strata or a thick single stratum, constrain the failure surface to a maximum depth of 1.2H. If all liquefiable strata are deeper than 1.2H, constrain the failure surface to the shallowest stratum.

Figure 2 Constraints on the failure surface

Restraining Action of the Foundation

While gravity and horizontal acceleration act to move the failure mass downslope, the bridge foundation acts as a restraining force as long as the piles are tipped into competent soil. In cases where the foundation consists of a small number of slender piles, the impact of its restraint may be modest. Conversely, large diameter shafts or piers may provide substantial restraint. Additionally, many case-histories have demonstrated significant restraining action by the bridge superstructure. If superstructure restraint is considered in the lateral spreading analysis, it is recommended that a global structural model be used.

The foundation's restraining action is accounted for by including a point resistance force along the slope's critical failure surface. The Restraining Force R_{Tot} for a single bent is shown in Figure 3. The foundation restraining force, R_{Tot}, is calculated by summing the individual resisting shear demands from each pile at the depth corresponding to the failure surface. The shear demands are checked against the pile's shear capacity (R_{max}) per SDC Section 3.6. Since slope stability analysis is performed per unit width, the point resistance force must be divided by an effective width of the failure mass. In the case of an abutment restraining an embankment, the effective width can be calculated as shown in Figure 4. If the abutment is located within a wide slope (i.e. no embankment) or in the case typical of interior bents where the width of the failure mass is much larger than the dimensions of the footing, the loading due to spreading ground extends beyond the immediate footprint of the foundation. For this case, the effective width can be taken as 1.5 times the foundation width.

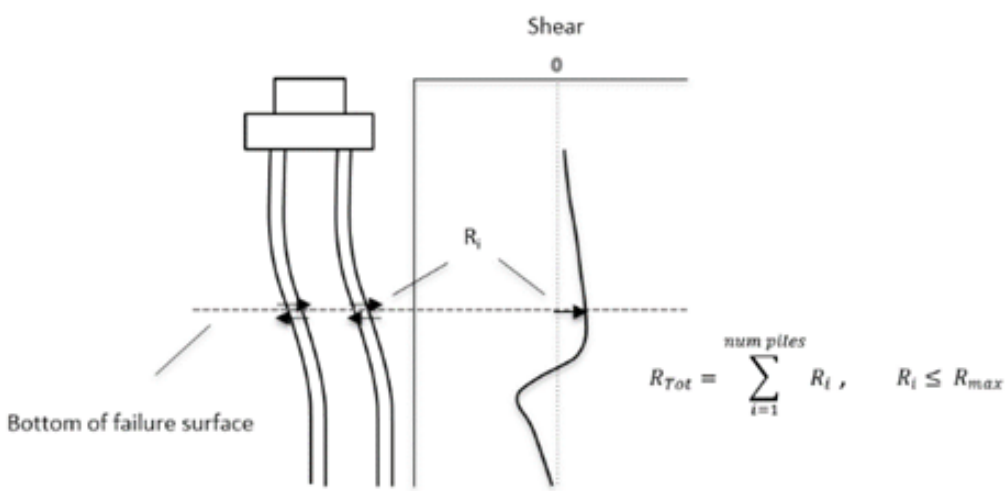


Figure 3 The foundation restraining force, R_{Tot}, is calculated by summing the individual resisting shear demands from each pile at the depth corresponding to the failure surface. The shear demands are checked against the pile's shear capacity (R_{max}) per SDC Section 3.6.

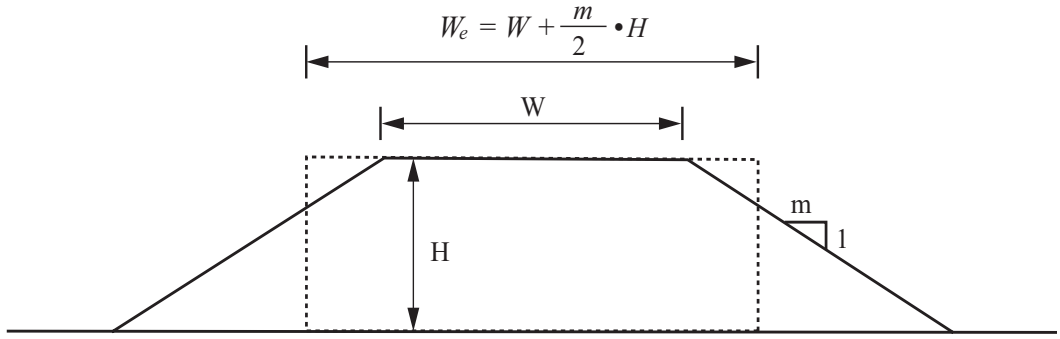


Figure 4 Effective width of embankment mass for abutment loading

Displacement Demand Calculation

The restraining action of the foundation will increase the yield coefficient of the failure mass, thus reducing the estimated displacement demand. The magnitude of the restraining force, however, is itself dependent upon the amount of slope movement. To account for this interplay between slope movement and restraining force, a method to calculate a compatible displacement is required. This method consists of the following steps:

1. (GD) Perform a slope stability analysis without consideration of the foundation restraining force to determine the depth of the critical failure surface.
2. (GD) Develop *p-y* curves for liquefied and unliquefied soil including those interacting with the pile cap, if applicable. Recommendations for *p-y* curve construction are provided in Attachment 1 of this memo.
3. (BD) Impose a range of increasing soil displacement profiles on the bridge foundation as shown in Figure 1(b). This analysis can be performed on a global model using finite element software or pile lateral load analysis program. Development of a global model will generally give better results. Only when the piles are uniform will a single pile model give results similar to a global bridge model. Recommendations for constructing an equivalent single pile model (for pile lateral load analysis program) are provided in the Attachment 1 of this memo.

   The displacement profile should be increased in increments until it reaches roughly 24 inches at the bottom of the failure surface (refer to Figure 3). For each increment calculate R_{Tot} as shown in Figure 3. Construct a plot of soil displacement (top of profile) vs. R_{Tot} as shown in curve (1) of Figure 5.
4. (BD) When the restraining force is applied in the slope stability model, it is applied as a static force. Since the restraining force is not static but varies with the level of displacement, an average value is used to approximate the static value. In cases where

curve (1) is significantly nonlinear, a better approximation is to use a running average. The running average at a displacement x_0 is given as follows, where R_i represents N roughly evenly spaced values of R ranging from $x = 0$ to $x = x_0$:

$$R(x_0) = \frac{1}{N}\sum_{i=1}^{N} R_i \tag{2}$$

Curve (2) in Figure 5 represents the running average restraining force. A value of N equal to 5 is considered sufficient.

5. (GD) Repeat slope stability analyses to calculate k_y for a range of restraining forces calculated in Step 4. Since the slope stability analysis is performed on a per unit width basis, the restraining force must be divided by the effective width of the foundation. For abutments the effective width is given in Figure 4. For interior bents, the effective width can be taken as 1.5 times the foundation width.

6. (GD or BD) The failure mass displacement can be estimated from ky using the following equation by Bray and Travasarou (2007):

$$D(cm) = \text{Exp}[-0.22-2.83\, Ln(k_y) - 0.333[Ln(k_y)]^2 + 0.566\, Ln(k_y)Ln(PGA) + 3.04\, Ln(PGA) - 0.244\, [Ln(PGA)]^2 + 0.278\, (M_w - 7)] \tag{3}$$

In equation (3), M_w is the magnitude of the design event. The PGA (in g's) and M_w should reflect a 5% in 50-year hazard level. Plot the calculated displacement vs. R_{Tot} as shown in curve (3) of Figure 5. Curve (3) represents the displacement response of the sliding mass shown in Figure 1. The intersection of curve (2) and curve (3) represents the design displacement.

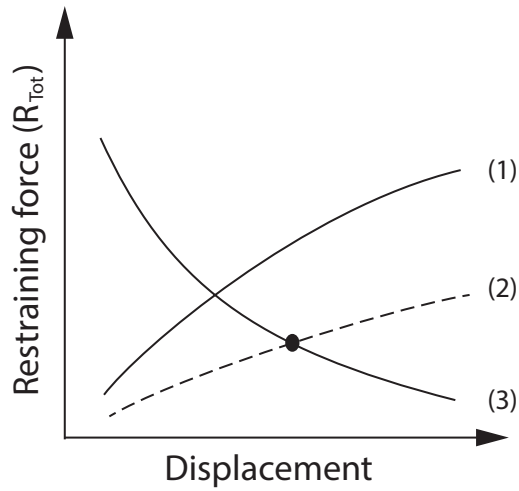


Figure 5 Interaction Curve (1) shows the total resisting force (R_{Tot}) vs. the foundation displacement. Curve (2) uses a running average of the resisting force to correct Curve (1). Curve (3) represents the displacement response of the sliding mass as calculated in Step 6. The design displacement demand is determined by the intersection of curves (2) and (3).

7. (BD) Load the foundation model with a soil displacement equal to the design displacement calculated in Step 6. The corresponding foundation displacement and structural demands must then be evaluated using the capacity estimation procedure specified in the "Capacity Estimation" of this memo.

   The restraining action of the superstructure can be accounted for (by the GD with input from the BD) in the above analysis by applying a point force in the slope stability as shown in Figure 6. The restraining action of the superstructure is represented by a point force acting at the lower 1/3 point of the abutment backwall. For seat type abutments, use only the breakaway portion of the backwall when applicable. For diaphragm abutments use the full abutment wall height. The point force should not exceed the passive resistance of the abutment reduced by a factor of two to reflect the average resistance as discussed in Step 4. The point force must be divided by the effective width of the abutment as given in Figure 4

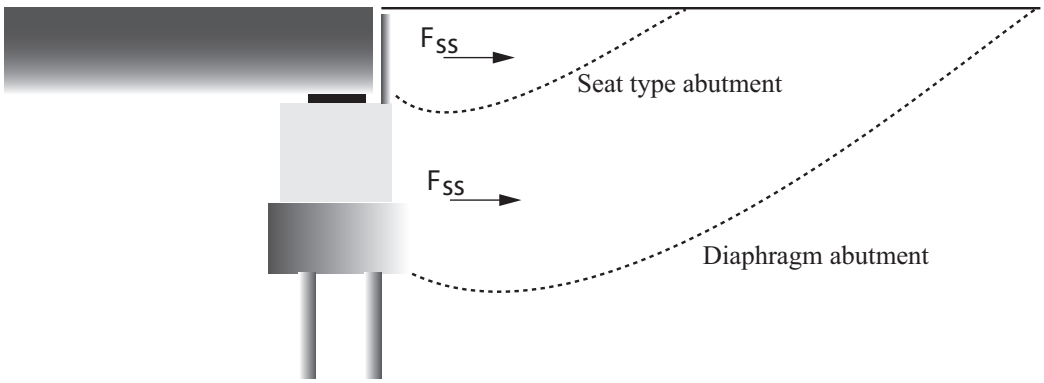


Figure 6 Superstructure Restraining Force F_{ss} at the Abutment

Capacity Estimation

Enhancements to Pile Capacity Due to the Surrounding Soil

Driven precast piles that are in liquefied soil but tipped into competent material have a significant displacement capacity. Although the piles may be damaged when lateral spreading occurs, the surrounding soil provides confinement, inhibits P-Δ effects, and allows the piles to continue carrying axial load. The piles can form two plastic hinges and both hinges can be degraded to pins and still support the bridge. This has been documented during several earthquakes. When the 12 m long, 350 mm diameter piles were exposed on the NHK Building (20 years after the Niigata earthquake) it was found that the piles had been severely damaged but had continued to support the building (Figure 7). Other examples of good pile performance in lateral spreading can be found in (Meyerson, 1992) and (Tazoh, 1996).

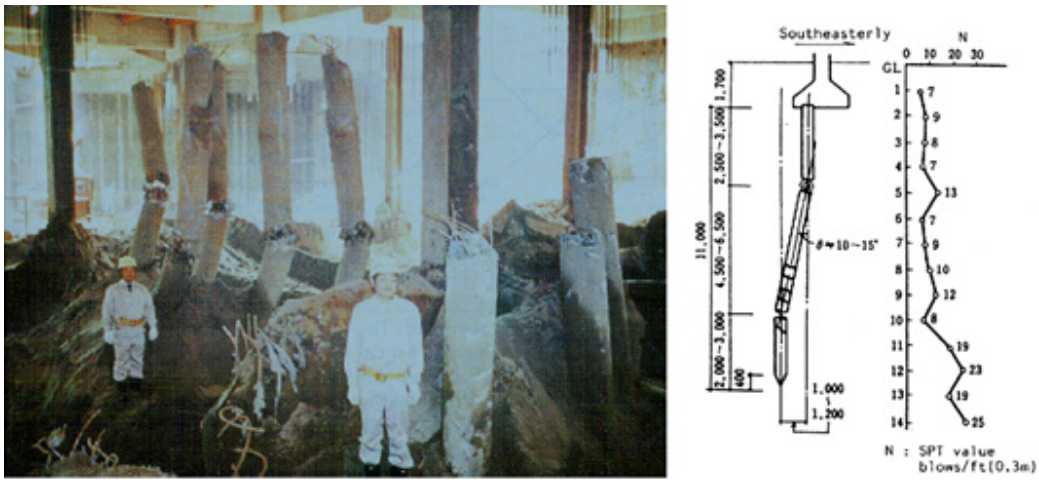


Figure 7 The NHK Building on piles that formed two pins during the 1964 Niigata EQ

Caltrans Office of Earthquake Engineering analysis shows that even poorly confined older piles have a drift capacity of about 15% before the ductility in both plastic hinges conservatively determined is exhausted. A drift capacity of 20% is possible as long as the surrounding soil in its liquefied state prevents buckling and *P*-Δ effects under permanent loads.

Conceptually, as the laterally spreading soil pushes against the piles (see Figure 8), the longitudinal reinforcement, prestressing tendons, and/or steel shell provides continuity as the plastic hinges degrade to pins. The crust and liquefied soil provide lateral support, the competent material provides axial support through skin friction and bearing, and the damaged piles can continue to provide stability to the bridge for permanent loads.

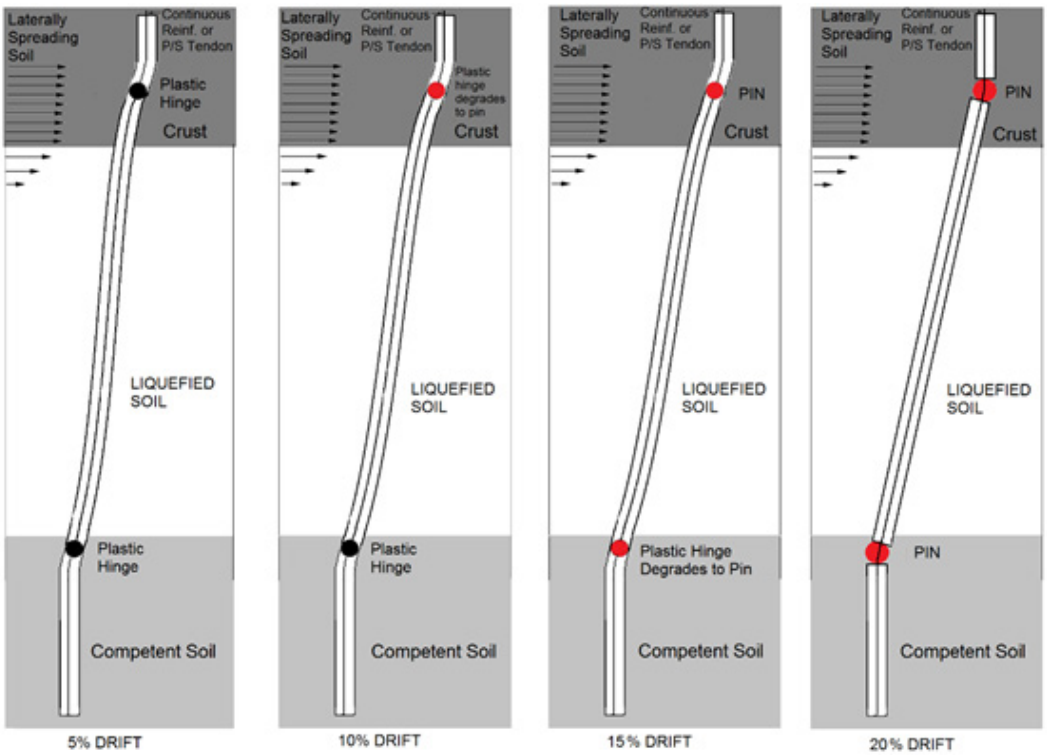


Figure 8 Lateral spreading of existing piles showing evolution from plastic hinges to pins

Lateral Capacity of Piles for Lateral Spreading Soil

New piles

Although piles in soil have a large displacement capacity as discussed in Section 3.1, the capacity of new piles is evaluated in terms of ductility. New piles are allowed to form two plastic hinges and may undergo ductility demands ($\Delta_{demand}/\Delta_{yield}$) up to 5.0. Large diameter piles and shafts like those shown in MTD 20-7 Seismic Design of Slab Bridges Appendix B (Caltrans, 2014) have good confinement and lateral capacity and are the preferred choice for new slab and T-girder bridges and should be more than adequate in poor soils. Type I and II Shafts provide the best lateral spreading resistance and ductility and should be used whenever possible in potentially liquefiable soil.

Existing Piles

Existing piles are allowed to be damaged or even to pin as long as the drift ratio ($\Delta_{demand/Leff}$) does not exceed the values given in Table 1. Old step tapered and cast-in-drilled hole (CIDH) piles are only allowed a drift ratio capacity of 0.05 unless the designer finds evidence that the casing or reinforcement can provide larger displacements. Analysis and testing has shown that timber (Shama, 2007) and precast piles have a drift ratio capacity of 0.20. Steel piles and especially steel pipe piles are good flexural members and can achieve drift ratio capacity of at least 0.20.

Although the shear capacity of piles is important when determining their plastic capacity, the post-plastic shear capacity does not need to be considered as the piles are just acting as an axial member supported by the surrounding soil. The related lateral capacity of columns and other bridge members can be obtained from Caltrans SDC (for new bridges) and from Caltrans MTD 20-4 (for existing bridges).

Table 1 Allowable Drift Capacity of Existing Piles and Shafts for Lateral Spreading

Element	Drift Ratio	Comments
Step Tapered in Steel Shell Piles	0.05	Larger capacity allowed through analysis
CIDH Piles with reinforcement only in top 10 ft of pile	0.05	Larger capacity allowed through analysis
Timber Piles	0.20	-
Driven Precast Piles	0.20	-
Steel Pipe Piles	0.20	Larger capacity allowed through analysis
Steel H Piles	0.20	Larger capacity allowed through analysis

Vertical Displacement Capacity for Existing Bridges

A bridge with a continuous superstructure is tough and resilient and can be severely damaged without collapse and without injury to people on the bridge. Most bridge foundations can displace vertically at least 24 inches without any danger of collapse. A two span bridge with a drop of 24 inches would only have a vertical offset of about 2% which most continuous superstructures can handle (Figure 9a). The exception is for bridges with discontinuous superstructures. A drop of 24 inches can be a risk to oncoming drivers and may be subject to unseating as the span rotates downward (Figure 9b). Bridge bearings can fall at an expansion joint causing a step that can injure oncoming drivers.

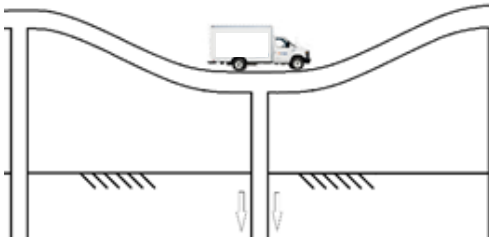


Figure 9a Continuous superstructure remains driveable if foundation drops

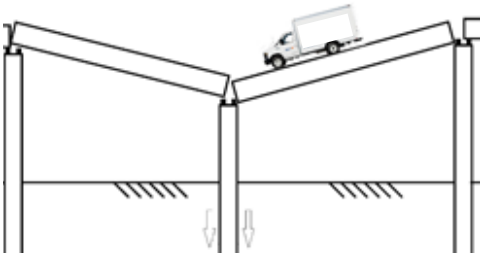


Figure 9b Discontinuous superstructure is a risk to the public

Based on the foregoing, for existing bridges the maximum allowable foundation settlement for continuous superstructures is 24 inches (2% of span length) and the maximum allowable settlement for discontinuous superstructures is 6 inches (0.5% of span length).

Bridge Retrofits

New and existing bridges are designed and retrofit for the same level of lateral spreading hazard. MTD 20-4 allows existing bridges more column displacement ductility (8 instead of 5) and more shear capacity. To contain costs, wherever possible the abutments and the superstructure are tied together to limit displacements of the bents. If additional piles are required, the BD should repeat Steps 3 and 4 in the "Displacement Demand Calculation" section of this memo to account for the modified foundation. Curve (2) in Figure 5 will then move upward, reducing the design displacement. This iterative approach is essential to achieving an economical strategy as described in MTD 20-4.

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Structure Policy and Innovation

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