# EFFECT OF BACKFILL SOIL TYPE ON STIFFNESS AND CAPACITY OF BRIDGE ABUTMENTS

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**ABSTRACT :**
Bridge abutments are earth-retaining structures which support the superstructure at the ends of a bridge and provide resistance to deformation and earthquake induced inertial forces from the bridge deck. Current design practice in California makes use of bi-linear load-deformation curve and does not account for the structure backfill properties. An experimental and an analytical research program were conducted at UCSD to further investigate such structure backfill interaction characteristics. The experimental program included five large-scale tests to examine the effect of structure backfill soil type, backfill height, vertical movement of the wall, and pre-existing cut slope in backfilling on stiffness and capacity of the abutments in the longitudinal direction. The study indicated that the response of bridge abutments in the longitudinal direction is nonlinear and a function of several influential factors which need to be considered. The results of the experimental program are presented in this paper.

**KEYWORDS:**
Bridge abutments, experimental testing, passive pressure, stiffness, and ultimate capacity.

## 1. INTRODUCTION

There are two types of bridge abutments generally used in state of California. Seat-type abutments support the bridge superstructure on a stemwall or "seat". Typically, short seat abutments include a backwall which retains the structure backfill material above the seat (Figure 1). Diaphragm abutments consist of an end diaphragm cast integrally with the superstructure and the abutment stemwall (Figure 2). The diaphragm supports the abutment approach fill under service conditions, and mobilizes the passive pressure longitudinally during seismic response. During an earthquake, the bridge superstructure moves longitudinally. Once the gap is closed, the superstructure collides with the backwall which induces deformation and inertial force to bridge abutments.

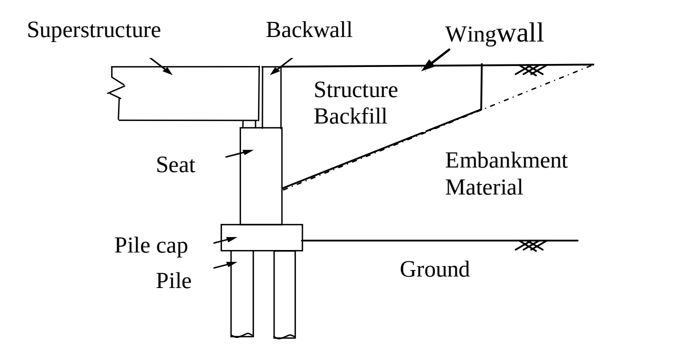



Figure 1.Bridge seat-type abutment, side view

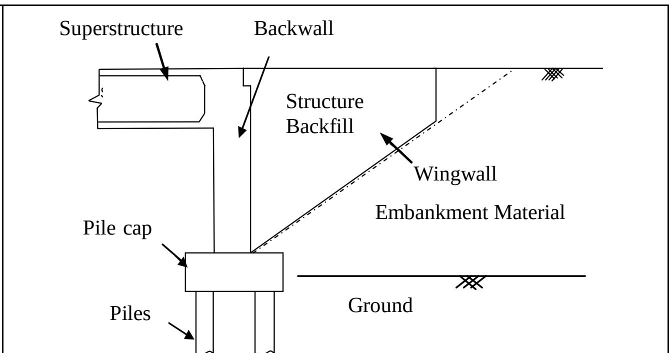



Figure 2. Bridge diaphragm abutment, side view

It should be noted that the current bridge design procedure (ATC 1996, SDC 2006) considers the backwall in seat-type abutments as a sacrificial element in order to protect abutment walls and piles from damage by limiting the inertial forces that can be transmitted into the abutment. Therefore, the force-resistance system of the bridge abutments in longitudinal direction during major seismic events is mainly provided by backwall-soil interaction, and the passive earth resistance behind the abutments. Neglecting the structure backfill properties in calculating the abutment stiffness and capacity results in predicting the abutment behavior unrealistically.

Several researchers have studied the passive earth resistance on pile caps, and retaining/abutment walls both experimentally and analytically. Wilson (1988) proposed a theoretical model for determining abutment stiffness based on the abutment dimensions and soil properties. Maroney (1995) tested two large scale diaphragm bridge abutments to failure at University of California, Davis. Martin et al. (1997) conducted advanced theoretical studies using a two-dimensional explicit finite difference computer program (FLAC) to characterize the load-deformation behavior of bridge abutments under cyclic loading. Shamsabadi et al. (2005) proposed a method to predict the mobilized force-displacement-capacity for the seismic design of a bridge abutment-embankment system.

Extensive experimental work associated with passive earth pressure has been conducted by several researchers such as Rowe and Peaker (1965); Narain et al. (1969); James and Bransby (1970); Fang et al. (1994 and 1997). There have been several large-scale tests conducted with interest in passive resistance recently (Romstad et al. (1995); Gadre (1997); Rollins and Sparks (2002); Duncan and Mokwa (2001); Rollins and Cole (2006)). The results of the literature review showed that only limited research has been done in the area of abutment capacity, and there is much uncertainty regarding appropriate modeling of bridge abutments.

## 2. PASSIVE RESISTANCE FORCE-DISPLACEMENT BEHAVIOR

A wide variety of methods are available to determine the capacity provided by passive pressure against the retaining structures. These methods include the classical approaches such as Log Spiral (Terzaghi 1943, Terzaghi et al. 1996), Rankine, and Coulomb, which are ultimate capacity predictors and do not capture stiffness behavior. In Rankine and Coulomb theory, it is assumed that the failure surface in the backfill is planar. However, Terzaghi (1943) showed that, due to the wall interface friction the real failure surface consists of a logarithmic spiral shape in lower part and a straight upper part.

The variation of passive resistance with displacement can be modeled by analytical expressions. Several methods have been proposed to characterize the development of passive pressure with displacement; including the Caltrans method, and the hyperbolic model given by Duncan and Mokwa (2001). The Caltrans method is based on the results from large scale abutments testing at University of California Davis (Maroney 1995). Caltrans (Seismic Design Criteria, 2006) suggests the initial longitudinal abutment stiffness to be equal to $20\frac{kip/in}{ft}$. The initial stiffness must be adjusted proportional to the backwall height as:

$$k_{abut} = k_i(= 20.0\ \frac{\text{kip/in}}{\text{ft}}) \times b \times \left(\frac{h_{abut}}{5.5}\right) \tag{2.1}$$

where, $b$ is the width of the backwall. The ultimate capacity of the abutment is given by Eqn. (2.2). The maximum passive resistance of 5 ksf in Eqn. (2.2) is based on the ultimate static force developed in large scale abutment testing at University of California, Davis. The height proportionality factor, $h_{abut}/5.5\,ft$, is based on the abutment test specimen height (5.5 ft) used at UC Davis (Maroney 1995). In Eqn. (2.2), $A_e$ is the effective abutment area. The passive pressure resisting the movement at the abutment increases linearly with the displacement, as shown in Figure 3.

$$P_{abut} = A_e \times 5.0 \;\; ksf \times \frac{h_{abut}}{5.5} \tag{2.2}$$

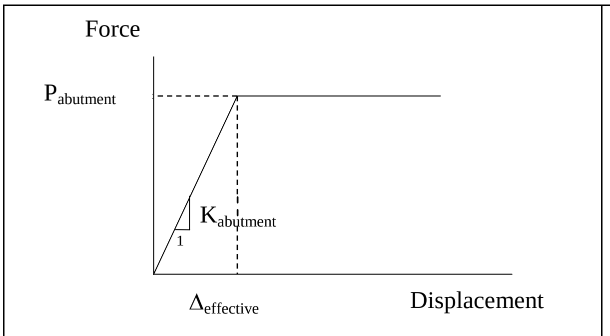



Figure 3. Force-displacement behavior of bridge abutment (SDC 2006)

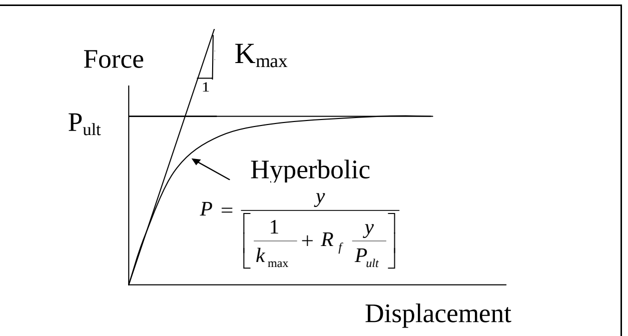



Figure 4. Hyperbolic passive force versus displacement (adapted from Duncan et al. 2001)

Figure 4 shows the hyperbolic representation of passive resistance-displacement relationship developed by Duncan and Mokwa (2001). The hyperbolic p-y curve developed by Duncan and Chang (1970) is expressed as:

$$P = \frac{y}{\dfrac{1}{k_{\max}} + R_f \dfrac{y}{P_{\max}}} \tag{2.3}$$

where $P$ is the load at any displacement $y$, $P_{max}$ is the ultimate passive force (using Log Spiral method); $k_{max}$ is the initial stiffness which corresponds to the initial slope of the load deflection curve. This value can be approximated using elasticity theory. The failure ratio, $R_f$, is defined as the ratio between the actual failure force and the hyperbolic ultimate force, which is an asymptotic value that is approached as $y$ approaches infinity. The value of $R_f$ can be estimated by substituting $P_{max}$ for $P$, and $\Delta_{\max}$ for $y$. Re-arranging the term in Eqn. (2.3) results in the following expression for $R_f$:

$$R_f = 1 - \frac{P_{\max}}{\Delta_{\max} k_{\max}} \tag{2.4}$$

The required movement to mobilize the maximum passive resistance, $\Delta_{\max}$, has been investigated by several researchers experimentally and numerically. Movement necessary to mobilize the maximum passive earth pressure suggested for different types of backfill are given in Table 2.1.

Table 2.1. Values of $\Delta_{max}/h$ for Different Backfill Soil Types (after Cole et al. 2006)

| **Backfill Soil Type** | **$\Delta_{\max}/h^*$** |
|---|---|
| Dense Sand | 0.01 |
| Medium-Dense Sand | 0.02 |
| Loose Sand | 0.04 |
| Compacted Silt | 0.02 |
| Compacted Lean Clay | 0.05 |
| Compacted Fat Clay | 0.05 |

*$\Delta_{\max}$ is usually expressed as a function of the height of the retained structure (h)

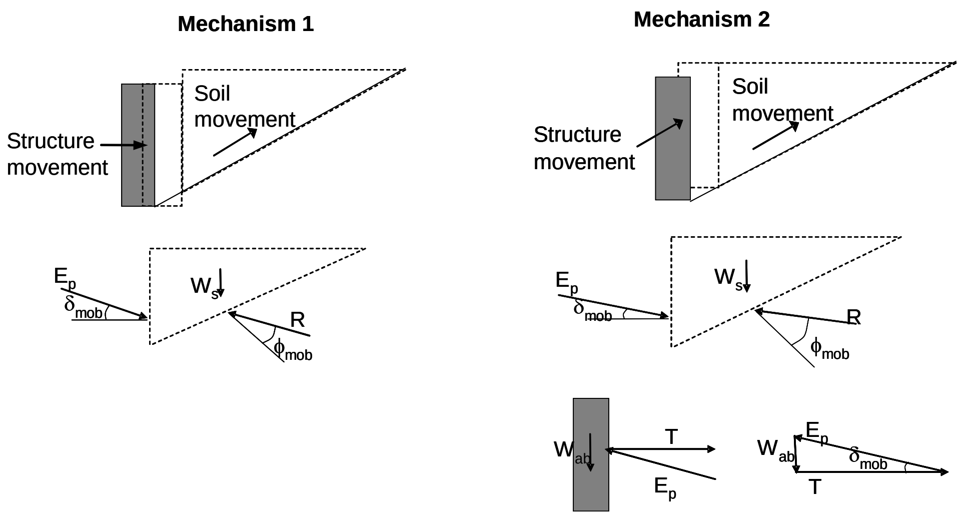
## 3. FACTORS THAT CONTROL THE PASSIVE RESISTANCE

One of the factors that controls the magnitude of the passive earth pressure that resists the movement of the wall is (Duncan and Mokwa 2001) the direction in which the wall moves. If the wall moves horizontally (vertical restraining force in the wall is greater than the vertical component of the passive pressure force), slip will occur on the interface between the structure and soil, and the value of the interface friction, $\delta$, will be controlled by the properties of the soil-structure interface (Figure 5a).

If the vertical restraining force in the wall is smaller than the vertical component of the passive pressure force, smaller relative displacement across the interface occurs which results in only partial mobilization of the interface friction (Figure 5b). The value of $\delta_{mob}$ must satisfy the vertical equilibrium as following:

$$W_{ab} = E_p \sin(\delta_{mob}) \tag{3.1}$$

where $\delta_{mob}$ is the mobilized friction angle, $W_{ab}$ is the weight of the structure, and $E_p$ is the developed passive pressure. As it can be noticed, the value of $\delta_{mob}$ is controlled by the requirements to satisfy the vertical equilibrium.




(a) (b)

Figure 5. Earth retaining structure and soil movements in passive pressure (after Duncan et al. 2001)

## 4. FIELD AND EXPERIMENTAL PROGRAM

In order to meet the objectives of the research project, both a field and experimental program were carried out. The objective of the field program was to develop a proper characterization of the soil types used for abutment structure backfills and its potential variation in the field (EMI Report 2005, Bozorgzadeh A. 2007). The result of the field investigation helped determine the type of structure backfill soils to be used for the tests properly. Two different types of soil were imported to be used as structure backfill in this research project, clayey sand and silty sand. Table 4.1 describes the Index properties and unit weight for each soil type. Because of geographic, time, and budget limitation in selecting the structure backfill materials, it was decided to consider just materials which provide the lower bound capacity and stiffness.

The test units were modeled to reflect typical diaphragm and seat-type bridge abutments commonly used in practice. The specific aims of the experimental program were to examine the effect of

1) structure backfill soil type
2) backfill height

3) restraining the vertical movement of the wall
4) pre-existing weak planes (pre-existing cut slope)

on stiffness and capacity of abutments in longitudinal direction. The bridge abutment tests were conducted at the field test facility from early January 2006 through June 2006.

Table 4.1. Summary of Index Test Results of Structure Backfill Materials

| **Soil Type** | **USCS**[a] | **Grain Size Distribution (Percentage Passing, %)** | | | **Dry Unit Weight**[c] **(pcf)** | **SE**[d] | **PI**[e] **(%)** |
|---|---|---|---|---|---|---|---|
| | | 75 mm | 4.75 mm | 74 μm[b] | | | |
| Clayey Sand | SC | 100 | 93-100 | 35-40 | 126.0 | 16-22 | 10-13 |
| Silty Sand | SM | 100 | 82-85 | 25-30 | 127.0 | 20-22 | <4 |

<u>Notes</u>: [a] ASTM D2487, [b] ASTM D1140, [c] ASTM D1557, [d] ASTM D2419, [e] ASTM D4318

## 5. TEST SETUP

The first phase of the experimental program was conducted on the performance of a component of a bridge abutment. An abutment wall (without a foundation) was built approximately at 50% scale of a prototype diaphragm abutment. The abutment wall was used for all four tests in Phase I. In this experiment, the desired failure mode was geotechnical, not structural. Therefore, the abutment wall was designed and built to remain in the elastic range during these tests. The key variables in Phase I were structure backfill type, backfill height, the area of structure backfill, and the vertical wall movement as shown in Table 5.1. The wall was restrained from rotational movement about three directions. Also, in Test 1, the wall was restrained vertically by means of proper configuration of actuators. The test setup was designed to model the longitudinal behavior of bridge abutments, restrained from translational and rotational movements. It was intended to follow the same construction phases of real bridge abutments in this project. The purpose was to study the influence of construction phases on bridge abutment behavior.

Table 5.1. Bridge Abutment Research Program Test Matrix, Phase I

| **Variables** | **Phase I** | | | | **Phase II** |
|---|---|---|---|---|---|
| | **Test 1** | **Test 2** | **Test 3** | **Test 4** | **System Test** |
| Soil Type | clayey sand | silty sand | silty sand | silty sand | silty sand |
| Structure Backfill Height | 5.5 ft | 5.5 ft | 7.5 ft | 5.5 ft | 5.0 ft |
| Structure Backfill Area | small | small | Large | large | large |
| Vertical Movement of Wall | restrained | allowed | allowed | allowed | allowed |

In Test 1, the setup of the actuators restrained any upward movement of the abutment wall to simulate the diaphragm abutment wall with fixed connection to its foundation (Figure 6). The actuators' setup in the rest of the tests in Phase I, allowed the vertical movement of the abutment wall to simulate the backwall sheared off from the stemwall in seat-type abutments. After Test 1, the structure backfill material was excavated and replaced by silty sand. The overall test setup for abutment Tests 2, 3, and 4 is depicted in Figure 6. It was decided to extend the excavation area for structure backfill in Test 3 and 4 to prevent any potential weak plane of failure in the test setup. The setup of actuators in Test 3 was similar to that in Test 2 which allowed the upward movement of the abutment wall. The height of the backfilling in Test 3 was 7.5 ft. In Test 4, like Test 3, the excavated area prior to placement of structure backfill was extended to the larger area. The only difference between Test 3 and Test 4 was the height of the backfill, changing from 7.5 ft to 5.5 ft.

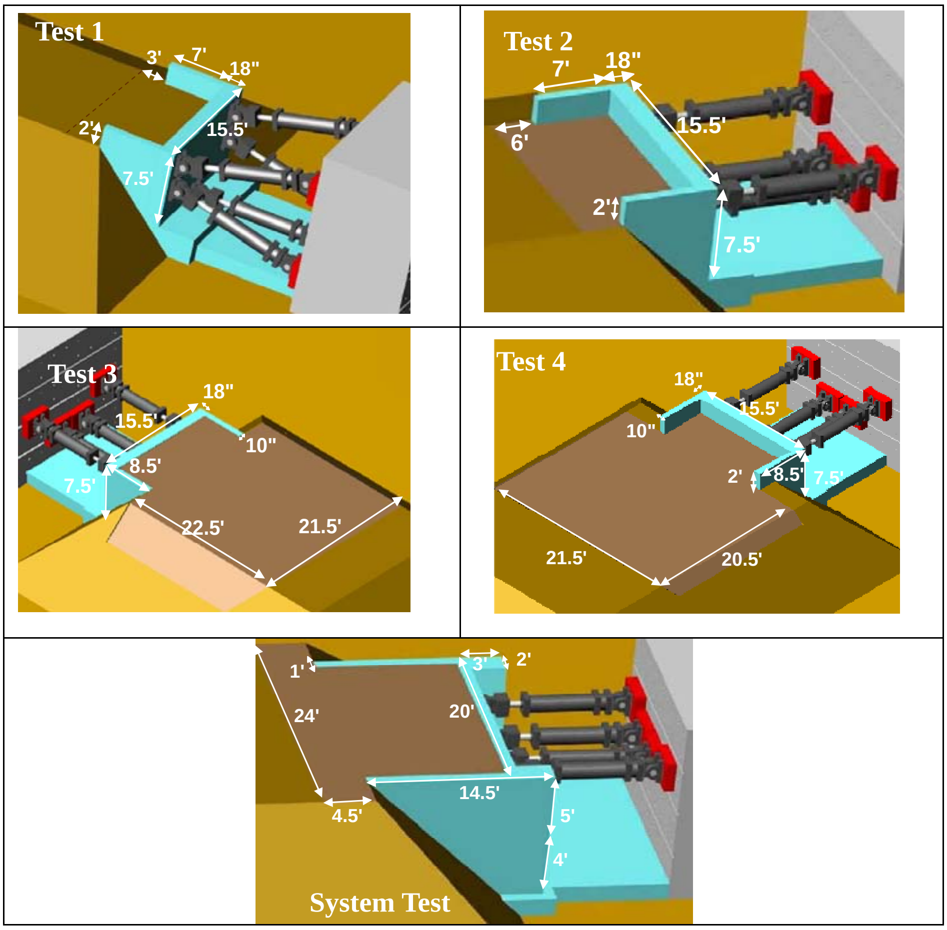



Figure 6. Overall test setup of the UCSD field abutment tests.

Although the setup for Tests 2, 3, and 4 in Phase I replicated the seat-type abutment behavior, the backwall of the test specimen was built monolithically with its wingwalls as a diaphragm abutment. Therefore, the second phase of this research program was conducted on a seat-type abutment which had a backwall separated from the seat (stemwall) and wingwalls. The abutment test unit consisting of the seat (stemwall), shear keys, wingwalls, and backwall was built at a large-scale of a prototype abutment. The overall test setup and design of test units are discussed in Bozorgzadeh (2007).

## 6. FORCE-DISPLACEMENT CURVE

The measured horizontal force-displacement response of the abutment wall tests is shown in Figure 7. The abutment wall in Test 1 was just allowed to move in horizontal direction, so the developed failure mechanism was the failure Mechanism 1 as described in section 3. The abutment force-displacement behavior was nonlinear up to the peak point; and after the peak it became approximately a horizontal line. The test was stopped after four inch displacement due to reaching the maximum capacity of two actuators. Figure 7 shows that the abutment in Test 1 was degrading with each cycle by comparing the loads at cycles with equal displacement peak. The permanent displacements at the end of each half loading cycle show the plastic behavior of the structure backfill soil.
Figure 7 shows the measured horizontal force-displacement response of the abutment wall Test 2, 3, 4, and System Test, respectively. The load test was under displacement control and performed monotonically. In test 2, 3, 4, and System Test, the abutment wall was free to move in the vertical

direction. The developed failure mechanism in these tests was the failure Mechanism 2 (Figure 5-b). The abutment force-displacement behavior was nonlinear during the tests. After reaching the peak point, the load started degrading. The inflection point occurred at approximately two times of the displacement at the maximum capacity. The tests were stopped after developing distinct failure cracks at the top surface of the structure backfill.

The force-displacement results indicate a substantial post-peak softening behavior in all the tests except in Test 1. The developed model to predict the force-displacement relationship of longitudinally loaded bridge abutments accurately is discussed in Bozorgzadeh, A. (2007). The following sections describe the effect of vertical movement of the wall, pre-existing cut slope in backfilling, backfill height, and the backfill soil type on the results of longitudinally loaded bridge abutment test units.

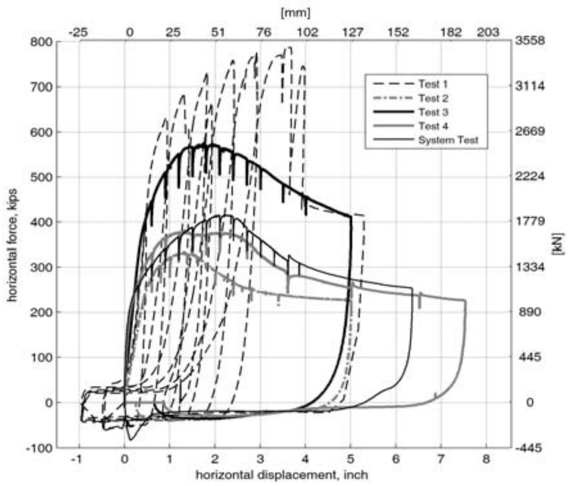



Figure 7. Horizontal force-displacement response, Phase I and II

## 7. EFFECT OF VERTICAL MOVEMENT OF THE WALL

If the wall movement restrained vertically (vertical restraining force in the wall is greater than the vertical component of the passive pressure force), slip will occur at the interface between the structure and soil, and the value of the interface friction, $\delta$, will be controlled by the properties of the soil-structure interface (Figure 5-a) as observed in Test 1 (diaphragm type abutment). The rest of the tests in Phase I and II were performed on the abutment wall which was free to move vertically (the backwall being sheared off from the stemwall in a seat-type abutment). The failure Mechanism 2 was developed in these tests where the smaller relative displacement across the interface occurred resulting in only partial mobilization of the interface friction (Figure 5(b)). The value of the $\delta$ has a considerable effect on the amount of maximum passive pressure (Coulomb theory and Log Spiral theory). Therefore, in cases where the interface friction is partially mobilized, the developed passive pressure is much less than the cases with fully mobilized interface friction.

## 8. EFFECT OF EXCAVATED AREA FOR STRUCTURE BACKFILL

Tests 2 and 4, in Phase I, were performed with two different structure backfill areas to evaluate the effect of structure backfill area on capacity of the abutment wall. All the variables, except the structure backfill area were kept the same in Test 2 and 4. In Test 2, excavated area was small, where in Test 4 the excavated zone was extended to a larger area. The maximum capacity of the abutment in Test 2 and 4 were not the same and was higher in Test 4 as shown in Figure 7. Therefore, the cut slope from the excavation in Test 2 introduced a weak plane of failure to the system and sliding occurred at that slope.

## 9. EFFECT OF BACKFILL HEIGHT

Tests 3 and 4 were performed with two different backfill heights to evaluate the effect of backfill height on capacity of the abutment wall. All the other variables were same. The ratio of the maximum passive pressure in Test 3 to the maximum passive pressure in Test 4 was calculated and compared with the ratio of the backfill height in Test 3 and Test 4. Figure 7 shows the force-displacement response of Test 3 and 4. The ratio of maximum passive pressure of corresponding tests is:

$$\frac{p_{\max,Test3}}{p_{\max,Test4}} = \frac{4.96}{4.48} = 1.11 \tag{9.1}$$

and the ratio of the backfill height is equal to:

$$\frac{h_{Test3}}{h_{Test4}} = \frac{7.5}{5.5} = 1.36 \tag{9.2}$$

The comparison between the Eqn. (9.1) and Eqn. (9.2) shows that the maximum passive pressure ratio is not equal to backfill height ratio. It is indicated that the passive pressure ratio is not directly proportional to the backfill height ratio. It is believed that considering the constant value for the maximum passive pressure and the stiffness then, corrected by the height proportionality factor will be poorly capable of calculating the maximum capacity and the stiffness of the bridge abutments with the same structure backfill and different heights.

## 10. CONCLUSIONS

The experimental research program was conducted to investigate abutment capacity and stiffness from field tests. Initially, a field investigation was conducted to investigate a proper characterization of the soil types used for structure backfill behind abutments and its potential variation in the field. The results from the field investigation showed a quite wide range of soil types have been used as structure backfill materials in bridge abutments in state of California.

A series of large-scale field tests were then performed at UCSD on bridge abutments loaded in the longitudinal direction. The results from testing program support the main theme of the research, that the response of bridge abutments is nonlinear to longitudinal load. Furthermore, the capacity and stiffness of bridge abutments depend on many factors which were studied during this research program to evaluate their influence. These factors are: soil properties, vertical wall movement, height of the backfill, and area of structure backfill. It was concluded that for the abutment backwalls designed to shear off during earthquakes, the post-peak softening behavior of the load-displacement curves should be considered in the soil spring model.

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