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SEISMIC RESPONSE AND CAPACITY EVALUATION OF EXTERIOR SACRIFICIAL SHEAR KEYS OF BRIDGE ABUTMENTS

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SUMMARY

The observed damage on bridge abutments and abutment piles after 1994 Northridge earthquake required a revision on the role and design of shear keys. Experimental research was conducted to investigate the seismic behavior of exterior shear keys that are designed in accordance to current guidelines. Experimental work was also performed on shear keys designed to act as a structural fuse in a bridge system that protect abutment piles from failure in a strong earthquake. In this paper, we discuss the results of the experimental program and development of a simple analytical model for capacity evaluation of exterior shear keys.

Keywords: abutments, bridges, experimental testing, sacrificial elements, shear keys.

INTRODUCTION

Shear keys are used in bridge abutments to support bridge superstructures transversely, as shown in Figure 1. In California, shear keys are designed as sacrificial elements that protect abutment walls and piles from severe damage by limiting the magnitude of transverse force transmitted into the abutment. Damage to abutments under a major seismic event is admissible provided that any abutment damage is repairable and there is no damage to the piles [1]. After the 1994 Northridge earthquake, extensive damage to bridge abutments was reported [2]. Large diagonal cracks on abutment walls were the common mode of abutment damage. Figure 2 shows an exterior shear key and an abutment stem wall damaged during the 1994 Northridge earthquake. Extensive damage to the abutment stem wall indicates that the exterior shear key did not perform as a structural fuse during the earthquake. The observed earthquake damage suggested a revision of current design methodologies for exterior shear keys. In philosophical terms, a shear key could transversely be designed as a sacrificial element to limit transverse inertial forces in the

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abutment walls and supporting piles. If shear keys are designed as sacrificial elements within a capacity design framework, their overstrength must be accurately determined to ensure other elements can be designed to remain elastic.

An experimental program was conducted at the University of California, San Diego, funded by the California Department of Transportation (Caltrans) to study the behavior of exterior shear keys under transverse forces. The main objectives of this research program were: (1) to reevaluate the validity of the design equation to estimate the capacity of sacrificial shear keys, (2) to provide data to develop an analytical model that could be used to estimate the shear key capacity accurately, and (3) to provide appropriate reinforcement detailing in shear keys which allow shear keys to perform as structural fuses.

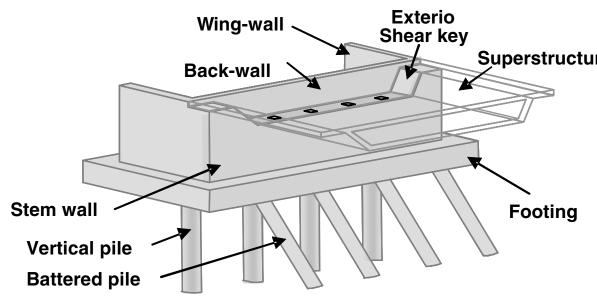

Figure 1. Schematic of exterior shear keys in bridge abutments, [3].

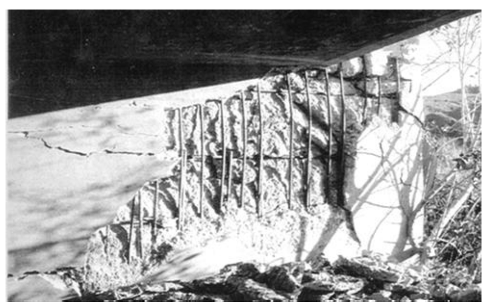

Figure 2. Abutment damage on South I-5/East SR-14, 1994 Northridge Earthquake, [4].

EXPERIMENTAL PROGRAM

Ten shear keys were built at 40% scale of a prototype abutment design provided by Caltrans and tested in five series during the program. Several factors were considered in this experimental program such as including construction joint between the abutment stem wall and the shear key, different amount and configuration of the vertical reinforcement crossing the abutment stem wall-shear key interface, and different amount and configuration of the horizontal reinforcement in the stem wall. Details for the amount of reinforcement and types of construction joint for each test unit are shown in Table 1. In Table 1, A_{sv} is the total area of vertical bars crossing the shear key-abutment stem wall and A_{sh} represents the total area of abutment horizontal reinforcing bars which transfer the shear force to the stem wall. In test series I, unit 1A was built without abutment back-wall and wing-walls, whereas unit 1B included both the abutment back-wall and wing-walls that were coupled together with the exterior shear key. In test series II, unit 2A was a redesign of unit 1A with the difference that in unit 1A the shear key was cast monolithically with the abutment stem wall, whereas in unit 2A the shear key was cast against the hardened and smooth concrete surface of the abutment stem wall. Test unit 2B was designed to display a more predominant flexural-shear response (see Figure 3). The design of the test units was based on results of previous test units with some changes to study the influence of changes on behavior of shear keys and the inertial force transferring mechanisms. For instance, test units 3A and 3B were redesigns of test unit 2A but included post-tensioning of the abutment stem wall in the transverse direction, with respect to the longitudinal axis of the superstructure. Also, shear key vertical reinforcement in test units 3A and 3B were located close to the center line of shear key. Smooth construction joint was provided for this test series (see Figure 4). Test units 4A and 4B represented the typical shear key design of Caltrans according to their Design Specification [1]. In test unit 4A, the shear key was built monolithically with the abutment stem wall while the shear key in test unit 4B was cast over a rough joint (Figure 5). All vertical reinforcement was continued from the abutment stem wall and was anchored to the shear key in test unit 4A. In test unit 4B, only the shear key vertical reinforcement (not temperature and shrinkage reinforcement) was continued to the shear key. Therefore, the amount of vertical reinforcement crossing shear key-stem wall interface was greater in test unit 4A than in test unit 4B.

Table 1. Test matrix of the five exterior shear key test series.

Test Series	Unit	A_{sv} (mm^2)	A_{sh} (mm^2)	Tie Reinforcement	Construction Joint
I	1A	2697	354.8	Hanger bars	None
	1B	2484	354.8	Hanger bars	None
II	2A	1703	425.8	Hanger bars	Smooth
	2B	3600	851.6	Hanger bars	None
III	3A	1600	1600	Headed bars	Smooth
	3B	1600	1600	Headed bars	Smooth
IV	4A	2910	1032	Hanger bars	None
	4B	1703	1032	Hanger bars	Rough
V	5A	516.1	1806	Headed bars	Foam*
	5B	516.1	1806	Headed bars	Smooth

*Foam with a center 203 mm by 203 mm cut out, placed at the interface of shear key- stem wall

Test units 5A and 5B had a reduced amount of shear key vertical reinforcement, A_{sv}. In test unit 5A, a 12.7 mm thick foam layer (with a center 203 mm by 203 mm cut out) was used as the shear interface between the shear key and the abutment stem wall. There was a smooth construction joint between the foam and the wall. In this test unit, the shear key was cast over a rough joint at the location of the hole. All shear key vertical reinforcement bars were lumped at one location in the rough construction joint, adjacent to the inclined face of the shear key. Test unit 5B had a smooth construction joint between the shear key and wall. A bond breaking film was applied to the construction joint in test unit 5B. All shear key vertical reinforcement bars were lumped at one location near the centerline of the shear key (4 #4 bars). Figure 6 shows the elevation and reinforcement details of exterior shear key units 5A and 5B. The tie reinforcement in test units 3A, 3B, 5A, and 5B consisted of headed bars. Hanger bars are referred to standard bars with 90-degree hooks at their end used for the tie reinforcement.

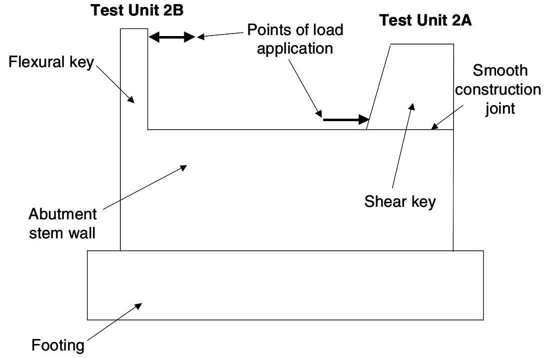

Figure 3. Exterior Shear key test series II, [5].

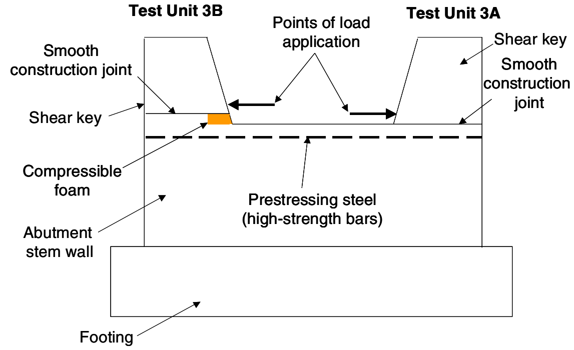

Figure 4. Exterior Shear key test series III, [5].

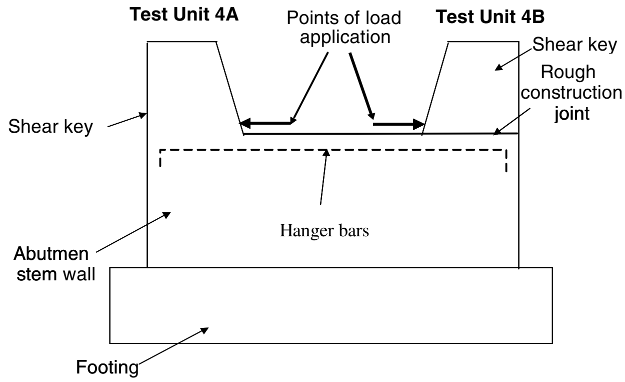

Figure 5. Exterior Shear key test series IV.

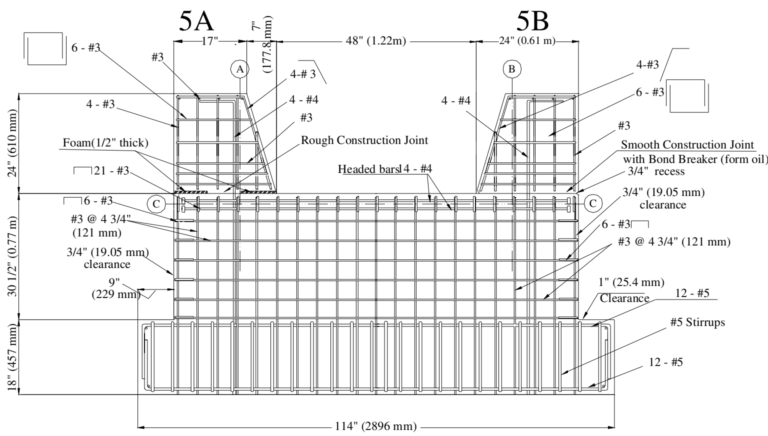

Figure 6. Elevation view of the reinforcement layout of test series V.

TEST SETUP

The test setup was designed to simulate the exterior shear key that interacts with the superstructure in a bridge during a seismic event. The abutment wall was post-tensioned to the laboratory strong floor. All test specimens were designed at a 2/5-scale with respect to a prototype abutment design provided by Caltrans. Each test unit was loaded by two horizontal 980 kN hydraulic actuators connected to a reaction wall (see Figure 7). The actuators were connected to a loading arm, which applied the lateral force to the test unit. A hold-down frame was used to prevent any upward movement of the loading arm. All test series had the same test setup except for test series V where the distance between the applied lateral load

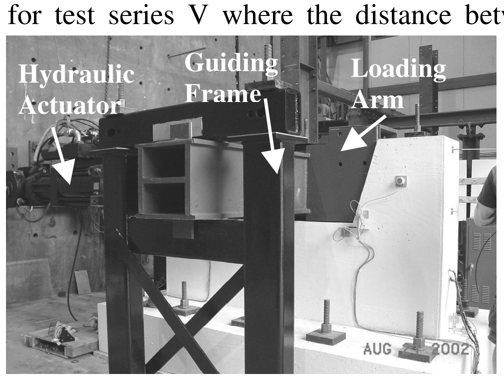

Figure 7. Overall test setup of exterior shear key.

and top of the stem was approximately 10 mm more than in all other test units. Also, in test unit 2B the force was applied at top of the shear key (Figure 3). The gap between the loading arm and each test unit was filled with 25 mm expanded polystyrene. In all test units, except test unit 2B, the loading was applied only in the push direction up to prescribed level of displacement, and then unloaded. Three cycles were performed at each displacement level. Test unit 2B was subjected to three fully reversed cycles at each displacement level up to failure.

GENERAL TEST OBSERVATION

Diagonal cracks initiated from the inner side of the shear key at the shear key-abutment stem wall interface during the first cycles in test units 1A, 1B, 2A, 4A, and 4B. As the tests continued, the cracks propagated diagonally to the toe of the stem wall. The first crack was the major crack during the tests and widened due to insufficient amount of shear reinforcement. The crack pattern observed in test unit 4B (with rough construction joint at the interface of shear key-stem wall) was similar to that observed in test unit 4A in which the shear key was cast monolithically with the abutment stem wall. A shear failure in the stem wall was observed in units 1A, 1B, 2A, 4A, and 4B (see Figure 8). At the end of the tests no shear sliding was observed in these test units. This indicates that exterior shear keys designed and detailed similar to these test units do not perform as sacrificial elements and, as a result, significant damage in abutment walls can be expected to occur under major seismic events.

Test unit 2B, with the flexural key, showed flexural-shear response. Ductile behavior was observed during the test of the flexural key. The flexural plastic hinge was formed at the shear key-abutment stem wall interface. Minor damage was observed in the abutment stem wall, which indicates that the flexural key of test unit 2B could prevent significant damage of the abutment stem wall (see Figure 9).

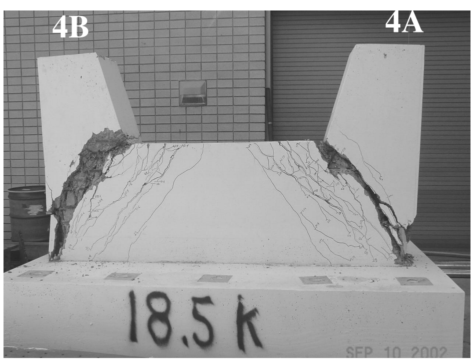

Figure 8. Test series IV after failure.

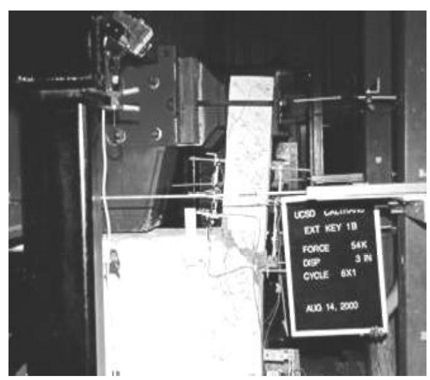

Figure 9. Failure of shear key test unit 2B, [5].

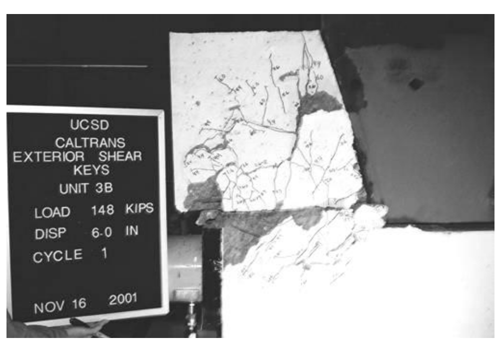

Figure 10. Failure of shear key test unit 3B, [5].

The formation of diagonal cracks in the abutment stem wall of test units 3A and 3B was prevented by applying a prestress force to the stem wall. Prestressing effectively precluded a diagonal shear failure. Instead, the failure mode in these two test units was shear sliding at the interface of the shear key-abutment stem wall followed by rupture of the vertical shear key reinforcement (see Figure 10). Test units 3A and 3B had a desirable sliding shear failure, but the capacity of these test units exceeded the maximum capacity defined by Caltrans [6].

In test unit 5A, sliding shear was observed after reaching the peak force. The shear strength of this unit was two times greater than predicted with shear friction models discussed by Walraven [7], and by Mattock [8]. Diagonal cracks formed in the abutment stem wall, however, the maximum width of the cracks was approximately 0.3 mm during the test. Figure 11 shows clearly that test unit 5A performed as a sacrificial element by shear sliding failure and prevented damage to the abutment stem wall. Test unit 5B achieved a shear sliding failure at the expected load. Only a very small crack was observed on the abutment stem wall with the width of less than 0.1 mm (Figure 12). Failure of both test units 5A and 5B occurred when vertical reinforcement of the shear key ruptured. Figure 13 shows this test series after failure.

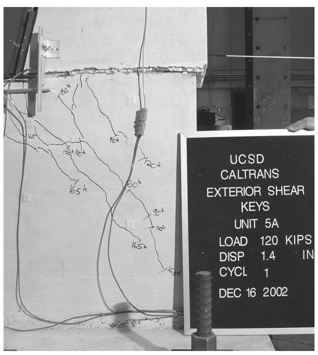

Figure 11. Failure of exterior shear key test unit 5A.

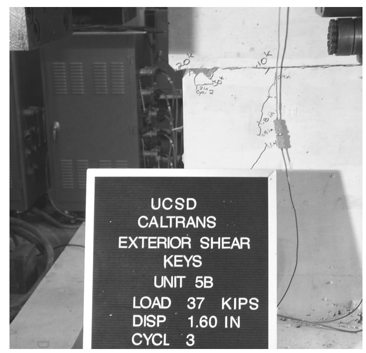

Figure 12. Failure of exterior shear key test unit 5B.

Figure 13. Test series V after failure.

LATERAL FORCE-DISPLACEMENT RESPONSE

The lateral force versus lateral displacement measured at top of the shear key test series IV and V are plotted in Figures 14 and 15, respectively. The reinforcement details of the abutment stem wall and shear keys of test series I, II, and IV were similar to those adopted in current practice in California. In test unit 4A, the first crack occurred at the lateral force of 445 kN, which was initiated at the interface between the shear key inclined face and the stem wall. The force capacity of the test unit 4A was 1,465 kN. The width of the major crack was around 10 mm when the peak force was reached. In test unit 4B, the first crack occurred at the lateral load of 391 kN. The force capacity of the test unit 4B was 1,330 kN. The width of the major crack was around 16 mm when the peak force was reached. The general response in both test units 4A and 4B was very similar. The failure mode in both test units was a diagonal shear failure in abutment stem wall. The capacity of these units was greater than the maximum capacity defined by Caltrans [6].

Figure 15 shows the force-displacement response of test units 5A and 5B. The response of test unit 5A shows an initially high stiffness. After reaching the maximum strength, a steep softening occurred in the response of this unit. As testing continued, a gradual increase in capacity, as a result of kinking of shear key vertical reinforcement, was observed. At a higher displacement the shear key vertical reinforcement ruptured followed by failure of the test unit. Test unit 5B performed as a sacrificial element by sliding between the shear key-abutment stem wall at predicted capacity. Figure 15 shows that the capacity dropped off at approximately 340 kN as the shear key vertical reinforcement reached the yield point.

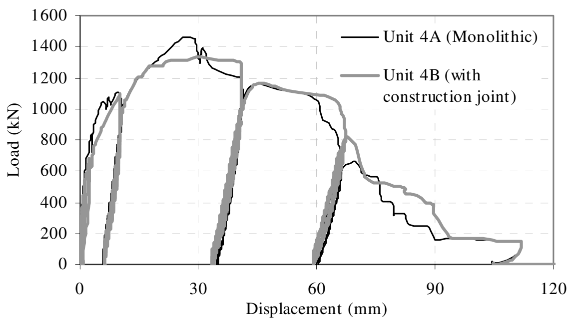

Figure 14. Lateral force-lateral displacement response of exterior shear key units 4A & 4B.

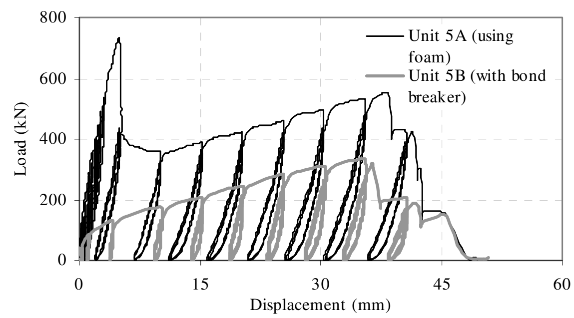

Figure 15. Lateral force-lateral displacement response of exterior shear key units 5A & 5B.

ANALYTICAL MODEL

According to the current Caltrans Bridge Design Specifications [1], the shear force capacity of shear keys, V_n, can be estimated by a shear friction model as follows:

$$V_n = \phi\, \mu\, A_{sv} f_y \quad \leq \quad 0.2\, f_c^{'}\, A_{cv} \tag{1}$$

In Eq. (1) ϕ is a strength reduction factor, μ is a coefficient of friction, A_{sv} is the area of vertical reinforcement crossing the shear key-abutment interface, f_y is the specified yield strength of reinforcement, f'_c is the specified concrete compressive strength, and A_{cv} is the area of the shear key-abutment interface. Table 2 presents the experimental shear force capacity of the exterior shear key units as well as their capacity calculated using Eq. (1) with $\phi = 1.0$. In test units 2A, and 4B (having construction joint at the interface of shear key-stem wall), Eq. (1) underestimates the capacity of shear keys. In test unit 2B, the significant difference between the capacity of the shear key calculated by Eq. (1) and measured during the test reveals that Caltrans shear friction model can not accurately predict the capacity of flexural shear keys. Also Table 2 indicates that in test series III and V (with shear sliding failure) the shear friction model substantially underestimates the capacity of shear keys. Based on these experiments, it seemed appropriate to develop an analytical model that could be used to estimate the shear strength of a shear key accurately. A strength evaluation of exterior shear keys was performed using strut-and-tie models. A mechanism model was developed for test unit 5B because this test unit performed as a sacrificial element with shear sliding failure at expected load. Figure 16 shows the developed model. The model took into account the deformed shape of the shear key. In order to measure the angle of kinked vertical bars, fractured vertical bars were removed from inside the shear key and stem

Table 2. Test results of exterior shear keys

Unit	f_c' (MPa)	V_{Test} (kN)	$V_{n,\ Eq\ (1)}$ (kN)	$\frac{V_{n,\ Eq\ (1)}}{V_{Test}}$
1A	34.2	988	1,690	1.7
1B	33.6	1,268	1,558	1.2
2A	21.4	707	592	0.8
2B	32.5	267	2,258	8.5
3A	38.8	1,188	476	0.4
3B	38.8	1,063	476	0.4
4A	39.8	1,465	1,716	1.2
4B	39.8	1,330	718	0.5
5A	33.6	736	224	0.3
5B	33.6	336	134	0.4

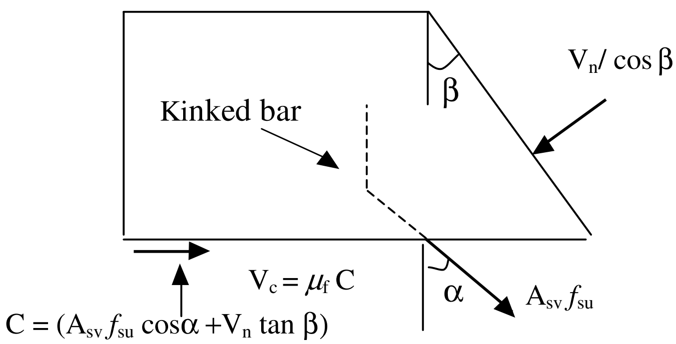

Figure 16. Mechanism model of exterior shear key in shear sliding failure, [9].

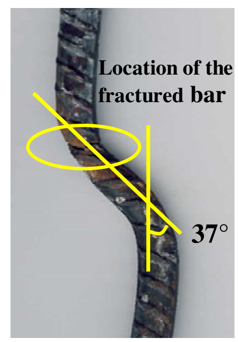

Figure 17. Fractured vertical bar in test unit 5B.

wall. Figure 17 shows one of the kinked vertical bars after assembling the two fractured pieces. By satisfying force equilibrium equations for this mechanism model, it is found that V_n is equal to 364 kN which is 8% greater than the shear force measured in the experiment for test unit 5B. The nominal capacity of shear key is given by:

$$V_n = \frac{\mu_f \cos\alpha + \sin\alpha}{1 - \mu_f \tan\beta} A_{sv} f_{su} \tag{2}$$

Where α is an angle of kinking of the vertical bars with respect to the vertical axis; β is an angle of inclined face of shear key with respect to the vertical axis (see Figure 16); μ_f is a kinematic coefficient of friction for concrete; A_{sv} is the amount of vertical reinforcement connecting the shear key to the abutment stem wall; and f_{su} is an ultimate tensile strength of the vertical reinforcement. Due to the kinematics of the sliding shear key, the vertical bars which connect the shear key to the stem wall must kink. Experimental works indicate the average kink angle, α, to be 37° at failure (Figure 17). By back-calculating the tensile force of vertical reinforcement and kink angle, α, from displacement data (measured during the test in unit 5B) and substituting in Eq. (2), the value of μ_f for concrete with smooth finishing is equal to 0.36. In test unit 5B, the ultimate tensile strength of the vertical reinforcement (#4 bars) was 710 MPa and the total area of vertical bars crossing the shear key-abutment stem wall was 516.1 mm^2. The Angle of inclined face of the shear key, β, in test unit 5B was equal to 16.3°. By substituting these values in Eq. (2), the nominal shear force capacity of unit 5B is equal to 364 kN. A smooth construction joint should be considered at the interface of the shear key-abutment stem wall, to effectively create a weaker plane at the shear key-abutment stem wall interface and enable occurrence of shear sliding at the interface.

Accuracy of the proposed model in predicting the capacity of shear keys is obtained by comparing analytical and experimental results. Table 3 shows the experimental shear capacity of exterior shear key test unit 5B, V_{test}, the shear force capacity according to the current Caltrans Specification, $V_{n,\ Eq\ (1)}$, and estimated capacity based on the proposed model, $V_{n,\ Eq\ (2)}$. Comparison of V_{test} and $V_{n,\ Eq\ (1)}$ reveals current shear capacity equation underestimates the capacity of the exterior shear keys, which may result in overloading and potential damage of the abutment piles during major seismic events.

Table 3. Computed and Calculated capacity of shear key test unit

Test Unit	V_{test} (kN)	$V_{n,\ Eq\ (1)}$ (kN)	$V_{n,\ Eq\ (2)}$ (kN)	$\frac{V_{n,\ Eq\ (1)}}{V_{test}}$	$\frac{V_{n,\ Eq\ (2)}}{V_{test}}$
5B	336	134	364.3	0.4	1.08

CONCLUSIONS

An experimental program to study the seismic behavior of shear keys was carried out at University of California, San Diego. Ten experiments were conducted on exterior shear keys. The results of the experimental work and analytical study conclude:

1. The current shear friction model underestimates shear key capacity. This could lead to damage of abutment walls or supporting piles under severe earthquakes before shear sliding failure at the shear key-abutment stem wall interface occurs.
2. A simple model for evaluation of capacity and behavior of shear keys under lateral force was developed. The model requires the use of kinematic coefficient of friction for concrete which is equal to 0.36 for concrete with smooth finishing .The experimental result from test was compared with the analytical result. The study concluded that the developed model for capacity evaluation of exterior shear keys has better agreement with the test results than the current shear friction model.
3. Abutments should have smooth construction joints between shear keys and the abutment stem wall to create a weak plane at the shear key-abutment stem wall interface. Shear key vertical reinforcement bars should be the only reinforcement connecting the shear key to the abutment stem wall. Abutment damage during major earthquakes can be considerably reduced by placing sufficient amount of horizontal reinforcement with good anchorage at both ends of steel ties in the stem wall.

ACKNOWLEDGMENTS

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