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AN INSIGHT INTO THE BAD REPUTATION OF BATTER PILES IN SEISMIC PERFORMANCE OF WHARVES

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ABSTRACT

In seismic areas, wharf and piers are susceptible to earthquake damages. These structures should be designed for an acceptable seismic performance. In some cases the use of batter piles both in designing new wharves and upgrading an existing wharf is inevitable. Not only do they control the displacement of the wharf, but also limit the number of plumb piles and decrease the cost. In spite of the advantages of batter piles, there are some problems associated with their usage. The paper presents the common problems of batter piles in a number of major earthquakes. The major problem of batter piles is tied to their seismic performance. In order to investigate this problem, a typical pier with and without batter piles is analyzed. The nonlinear static procedure (pushover), which is an acceptable method for analyzing systems with a single degree of freedom, is used. The analytical studies show that the lateral stiffness of the pier considerably increases by the use of batter piles; which leads in a higher seismic demand. Moreover, since batter piles are subjected to compressive axial loads, their ductility is widely questioned. Another problem associated with the use of batter piles is the concentration of high forces leading in major damages in pile cap and/or pile head. Based on the insight gained in this research, it is recommended to avoid using vertical and batter piles together. However, if the use of batter piles is essentially inevitable, attention should be paid to proposed considerations and solutions presented in the article.

Keywords: Batter Piles, Pier, Wharf, Seismic, Damage, Pushover

INTRODUCTION

From amongst the different alternatives for designing new wharves and piers, the pile supported systems have been one of the most common choices. For example, pile supported structures are the most prevalent kind particularly on the US west coast.

Plumb piles have shown to be proper for resisting vertical loads; however, they have a limited capacity for supporting the lateral loads such as berthing, wind, and earthquakes load. As a result, if a design consists of only plumb piles, the lateral displacement would be considerably high in most of the cases. Thus, satisfying the criteria for limiting horizontal displacement would lead in designing large size piles. Hence, using the flexural resisting members without benefiting from any other members with axial strength will definitely be uneconomical. Up to now, various methods have been proposed for increasing the lateral capacity of the piers. One of the most effective and common methods is usage of batter piles.

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However based on the observations from the past earthquakes, the simultaneous use of the batter pile and plumb pile has been found to be a challenging debate. Due to poor performance in past earthquakes, the use of batter piles to resist lateral loads under piers and wharves is on the decline. According to Marine Oil Terminal Engineering and Maintenance Standard (MOTEMS 2003) which uses a performance-based methodology for seismic design, lateral systems making use of batter piles are discouraged unless special studies are conducted that show such a system can perform within the limits of the criteria.

In this research, the common problem of batter piles in recent earthquakes has been recognized. An insight to seismic performance of these structures is gained by an analytical study of a typical pier with and without batter piles. The analysis described in this paper was performed using SAP2000 Nonlinear version 8.3.1. Finally, based on the findings of this research, some suggestions are proposed to reasonably use the batter pile and plumb pile simultaneously in a design.

BATTER PILE DAMAGES DURING EARTHQUAKES

Assessment of the observed seismic performance of wharves and piers during past earthquakes can provide valuable insight for recognition of the problems associated with batter piles. A relatively thorough study has been carried out in this respect by the authors; however, we will only touch on three important earthquakes in this part.

Alaska Earthquake of 1964

The 8.3 magnitude Alaska Earthquake caused extensive damage to highway bridges. The City Dock suffered extensive damage, although it was a modern reinforced concrete structure supported on 40 to 110 cm diameter steel pipe piles and 35 to 50 cm diameter battered piles driven into stiff “Bootlegger Cove” clay. Margasson (1997) estimated that the peak horizontal ground acceleration (MHA) at this site was 0.30 g, and observed that the added inertial loads due to the dock being heavily loaded with ice aggravated the seismic response. The batter piles battered to the west were bowed and buckled while those battered to the east were relatively straight. The evidence was that the displacement of the pier, both translational and rotational, was developed and retained by buckling of the batter piles in the west portion of the main pier. The working of these batter piles against the large horizontal seismic forces indicated vertical loads at the pile heads which were transmitted into concrete pile caps above. The seismic forces further caused a swaying of the pier in a rotational and east-west direction, inducing bending moments and shears at the tops of the vertical piles. In some cases these stresses caused shattering of the concrete cap and deck around the pile head. The deficient response of battered piles was particularly evident in the manner they attracted horizontal load and transmitted these loads into the structure and other piles.

The Loma Prieta Earthquake of 1989

The 7.0 magnitude Loma Prieta Earthquake caused the dramatic failure of several pile-supported structures. Port facilities and marine structures around San Francisco Bay experienced moderate pile damage in the earthquake (SEAOC, 1991). At the Port of Oakland, a MHA of 0.45 g was measured. The 7th Street Terminal Complex suffered extensive damage, as 40 cm square prestressed concrete batter piles supporting the Public Container Wharf failed in tension at their connection to the deck (Figure 1). About 600 or 95 percent of the batter piles at the wharves along the seventh street Terminal experienced cracking and fracturing, usually at their connections to pile caps (Port of Oakland, 1990). The Matson Terminal Wharf at 7th Street suffered similar damage, but with additional damage to the back row of vertical piles. At the Oakland Outer Harbor Pier 7, 40 cm square prestressed concrete batter piles failed at or near the connection to the pile cap. In San Francisco, the Ferry Plaza Pier experienced tensile failure at the connection of the deck to the prestressed concrete batter piles, with some of the piles punching the slab. Spalling and cracking of the bottom of the slab was found at over

100 pile locations. At Piers 27 and 29, similar damage as that experienced at the Ferry Plaza occurred to over 120 50 cm square prestressed concrete batter piles (Figure 2).

Generally, damage to batter piles was often due soil liquefaction, settlements and lateral soil spreading. The damage occurred at the tops of the batter piles through shear, compression, and tension. The vertical piles were largely undamaged with a few exceptions. The stiff batter piles absorbed much of the loading among the other more flexible elements.

As a result of this damage, the port of Oakland replaced all the 7th Street Terminal batter piles with vertical piles designed to resist lateral forces. The Howard Terminal and the APL Terminal which had vertical or near vertical piles instead of batter piles did not sustain pile damage.

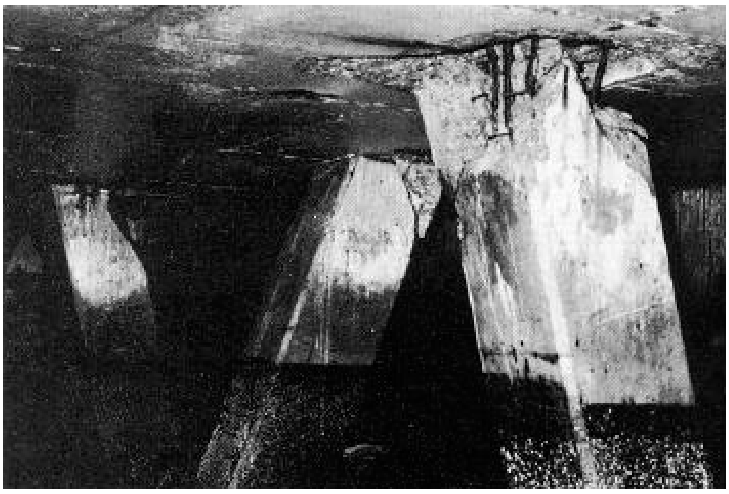


Figure 1. Damaged Batter Piles at Port of Oakland 7th Street Terminal during the1989 Loma Prieta Earthquake (after SEAOC, 1991)

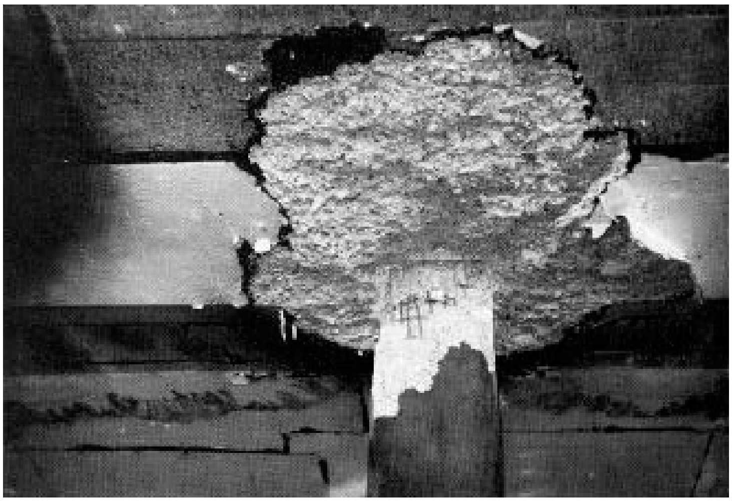


Figure 2. Damaged Batter Piles at Port of San Francisco Piers 27 & 29 during the 1989 Loma Prieta Earthquake (after SEAOC, 1991)

The Costa Rica Earthquake of 1991

The 7.5 magnitude Costa Rica Earthquake caused severe damage over a large area, including collapse of several pile-supported bridges. The south abutment rotated about 9 degrees, causing movement of the

36 cm square precast concrete piles 66 cm toward the river (Figure 3). The front battered piles suffered flexural and shear damage, but the vertical piles at the rear showed less damage (Figure 4).

Figure 3. Preferential Damage to Front Batter Piles of Rio Banano Bridge during the 1991 Costa Rican Earthquake (after Priestly et al., 1991)

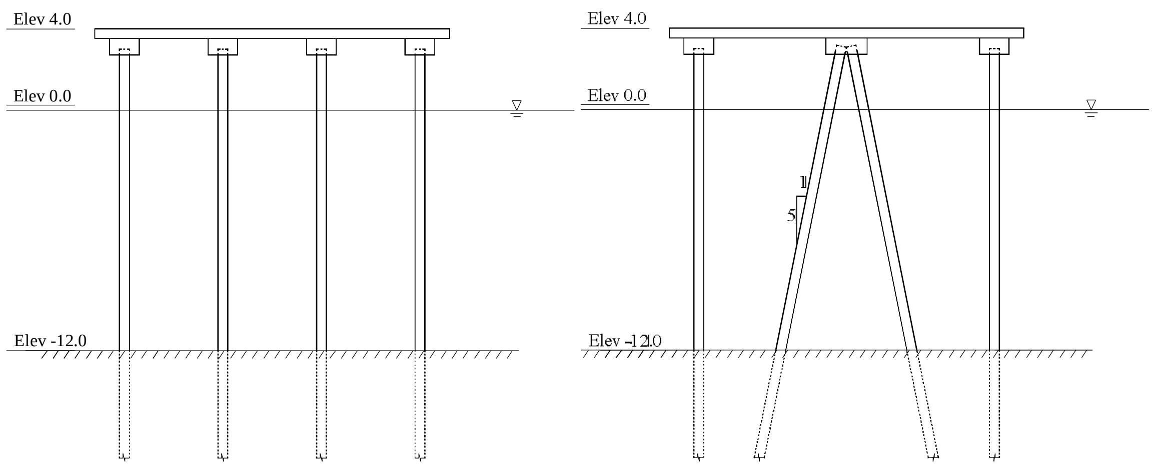
Figure 4. Failure of Rio Viscaya Bridge Piles during the Costa Rican Earthquake (after Priestly et al., 1991)

Similar damages have also occurred during other earthquakes such as Tokachi-Oki, Japan 1968, Miyagi-Ken-Oki, Japan 1978, San Fernando 1991, Guam 1993, Northridge 1994, Manzanillo, and Mexico 1995.

Therefore it is concluded that, batter piles at piers and wharves have often performed poorly during past earthquakes. This is due to the large lateral stiffness of the batter piles, which mobilizes large lateral seismic forces at pile cap and decking connection. This has led to severe cracking, fracture, and failure of the piles, and major damage to the pile cap and decking. Batter piles and their connections have typically been designed to resist axial loads only. Their design has not provided adequate strength and ductility to resist the lateral deformations, bending moments, and shear forces caused by lateral seismic excitations.

3D STRUCTURAL MODELING

In order to investigate the performance of batter piles in piers, a typical pier has been modeled and analyzed. The pier deck is 15m x 15m supported on 16 steel pipe piles. The piles are driven to the depth of 20 m in a sandy soil. The distance between two adjacent piles in frame is 5 m centre to centre. The deck is composed of a reinforced concrete with connecting concrete beams. The piles are connected to the deck through the pile caps which make a rigid connection. Pier B is different from pier A in terms of the number of plumb piles, i.e. in pier B, two vertical piles in each side frames are replaced with batter pile. (Figure 5)



Dense Sandy Soil, $\varphi' = 40?$, $\gamma = 18\ kN/m^3$, $C' = 0.0$

a) Pier A: without batter pile **b) Pier B: with batter pile**

Figure 5. Pier layout and soil parameters

The nonlinear static procedure, pushover method (FEMA 356, ATC 40), was used to perform the analysis. Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. With the increase in the magnitude of the loading, weak links and failure modes of the structure are found. The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated using a modified monotonic force-deformation criteria and with damping approximations. The pushover curve (capacity curve) gives a good prediction of failure mechanisms and accounts for redistribution of forces during progressive yielding. SAP2000 is capable of calculating the performance point for a certain demand spectrum. The performance point represents the maximum structural displacement or acceleration expected for the demand earthquake ground motion.

Besides the relative simplicity of pushover method compared to the nonlinear dynamic procedure and the good approximation of the seismic behavior of the structure, this method has some disadvantages. Generally, the disadvantage of pushover method is associated with its static nature. Accordingly, where the dynamic effects govern the behavior of the structure such as structures with multi degrees of freedom, pushover method does not predict the total behavior properly. According to FEMA 356 pushover shall be permitted for structures in which higher mode effects are not significant. Thus for single degree of freedom (sdof) structures, such as piers and wharves, it is an acceptable method. On this basis “MOTMES” and “Seismic Criteria for California Marine Oil Terminals” have recommended this method for analyzing wharves and piers.

The deck was modeled as a thick shell element 50 cm in depth and the piles were modeled as frame element 50 cm in diameter and 1 cm in thickness. Concrete and steel material properties used to model the deck and piles are listed in table 1.

Table 1. Material properties used in the model

Concrete Unit Weight	Concrete Compressive Strength	Steel Unit Weight	Steel Yield Strength	Steel Ultimate Strength
$\gamma_c = 24$ kN/m^3	$f'_c = 25$ MPa	$\gamma_s = 78.5$ kN/m^3	$f_y = 360$ MPa	$f_u = 450$ MPa

The Overseas Coastal Area Development Institute of Japan (OCDI 2002) allows a virtual fixed point, located below the sea bottom, to be assumed in order to consider the end fixity of the piles for analyzing a wharf. In order to model soil-pile interaction more concisely, a series of elastic-plastic nonlinear link (Multi Linear Link) elements was used. MLLink elements allow for an elastic spring stiffness and nonlinear behavior of the soil. Spring force-deformation relation including p-y, t-z, and Q-z (for end springs) was developed according to American Petroleum Institute (API, 2000) method. The spring characteristics were calculated for increments of 1 m for the first 5 m below the surface and continued at every 2 m to the depth of 20 m, resulting in thirteen different MLLink elements. Ferritto et al. (1999) recommend that the most accurate and easiest method to model the soil-pile interaction for the batter piles is the one with spring elements axial and normal to the batter pile. The soil geotechnical characteristics are shown on Figure 5. Figure 6 shows the 3D model of pier B.

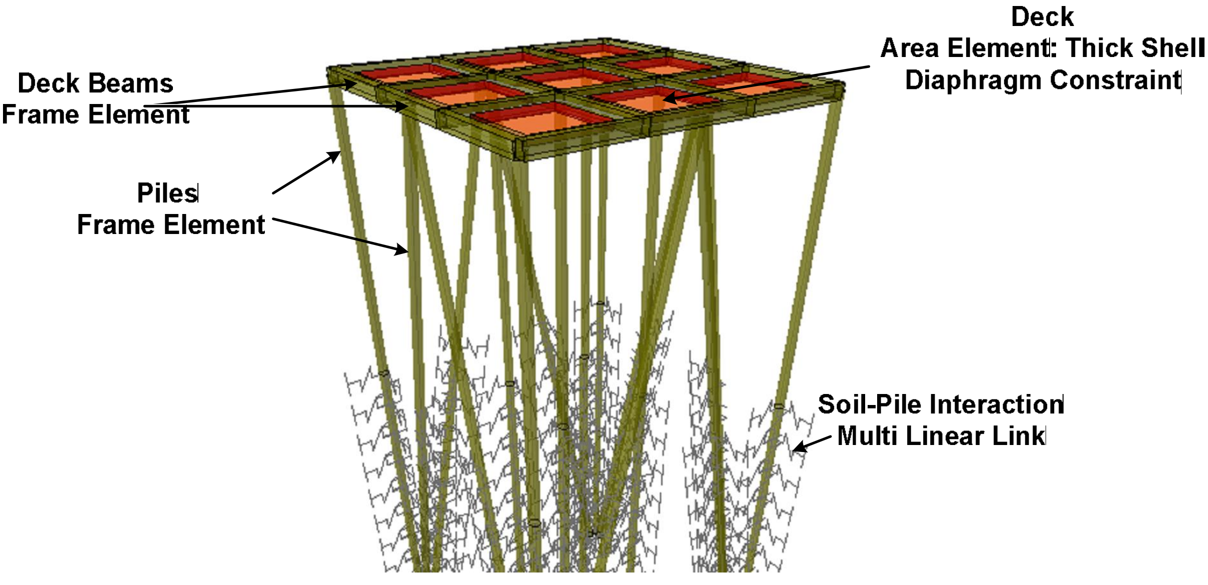


Figure 6. Three D Structural Model of Pier B

DISCUSSION

The pushover analysis shows that pier A can resists 131 tons applied horizontally before the formation of the first plastic hinge. The maximum displacement (S_d) of the pier due to a certain demand spectrum was found to be 0.082m. This displacement is corresponding with the maximum acceleration (S_a) of 0.484g. The first plastic hinge was formed at top of the piles at the deck level and a second hinge developed at the sea bed. The structure underwent large gradual deformation near failure.

As for Pier B, in which 25 percent (4 out of 16) of its vertical piles were substituted by batter piles in each direction (Figure 5), the lateral stiffness was considerably increased. The initial stiffness of the pier was 1618 ton/m in comparison to 456 ton/m of pier A. The maximum displacement (S_d) was 0.039 m (about half of pier A) which is corresponding with the maximum acceleration (S_a) of 0.780g. The first plastic hinge was formed in a batter pile. At failure, the load was transferred to the remaining batter pile, which itself failed causing failure of all the remaining piles. The structure was observed to perform in a brittle manner.

As shown in Figure 7 and 8, the capacity curve of pier A is smooth and uniform, while the capacity curve for pier B is sawtooth with sudden drops at the beginning. The drops represent the failure of the batter piles. The pushover curve of the two piers shows the relatively ductile behavior of Pier A in comparison with the brittle behavior of pier B. The axially loaded members are essentially incapable of having high levels of ductility. The same rule applies for the batter piles which support considerable amounts of vertical loads, which results in a less ductile pier. The AASHTO (2002) recommends considering a response modification factor of 5 for designing piers including plumb piles, and R=3 for piers including batter piles, which confirms the findings of this study. A solution to this problem has been posed by the authors in the last part of this article. Table 2 presents a summary of the results of analysis for pier A and B.

Table 2. Results of analysis for pier A and B

Pier	Batter Pile	Period (s)	K_{eff} (ton/m	S_a	S_d (m)	V_{max} (ton)
A	No	2.65	456	0.484g	0.082	244
B	Yes	1.01	1618	0.780g	0.039	262

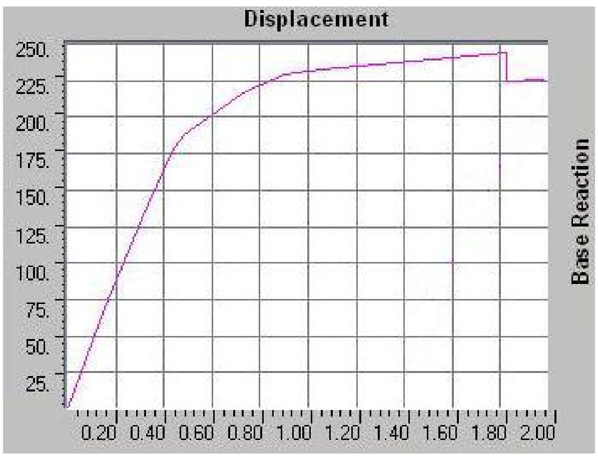


Figure 7. Capacity curve of pier A

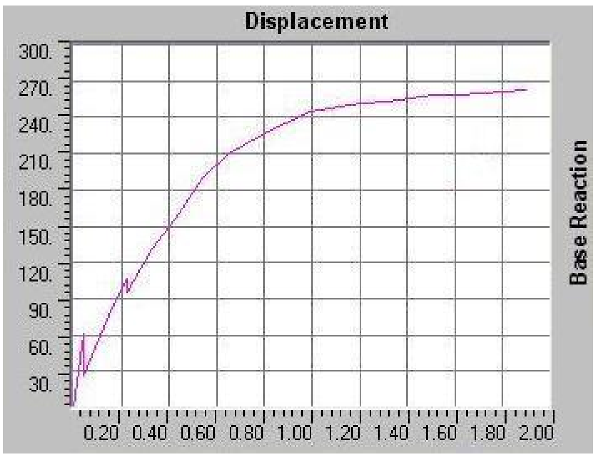


Figure 8. Capacity curve of pier B

CONCLUSION AND RECOMMENDATIONS

As a conclusion, the results of analysis can be summarized as follows:

- The high lateral stiffness of batter piles increases the lateral stiffness of pier, as well as the demand of the pier, but reduces displacement.
- The batter piles are the stiffest members in the structure and thus absorb a high portion of the exerted load which results in their failure. Once a batter pile is fails the force is redistributed in the structure and may cause subsequent failure of other members.
- The most common failure of batter piles occurs at the connection with the deck.
- The piers supported on batter piles have less ductility than the pier without batter piles.

Based on the findings in previous parts, it is recommended to avoid the use of batter piles with plumb piles in a design. If the use of batter piles is inevitable, the followings suggestions are proposed by the authors:

1- Do not connect a batter pile to a plumb pile in a joint. The forces will be concentrated in the batter pile and cause the failure of batter pile. The failure of the batter pile would simply extend to the plumb pile and consequently cause the failure of the structure.

2- Assign the vertical loads to vertical piles, and lateral loads to the batter piles. It helps to design a more ductile structure.

3- Apply a structural fuse in the structure which entails advantages such as increasing the ductility of the structure, absorbing a greater amount of energy, concentrating the damage in a the fuse, and preventing other members to undergo significant damage.

In Figure 9 the deformation pattern of the pier under horizontal seismic load is depicted. Figure 10 shows a proposed system which considers all the suggestions made earlier. The proposed suggestions will result in economical and well performing structures.

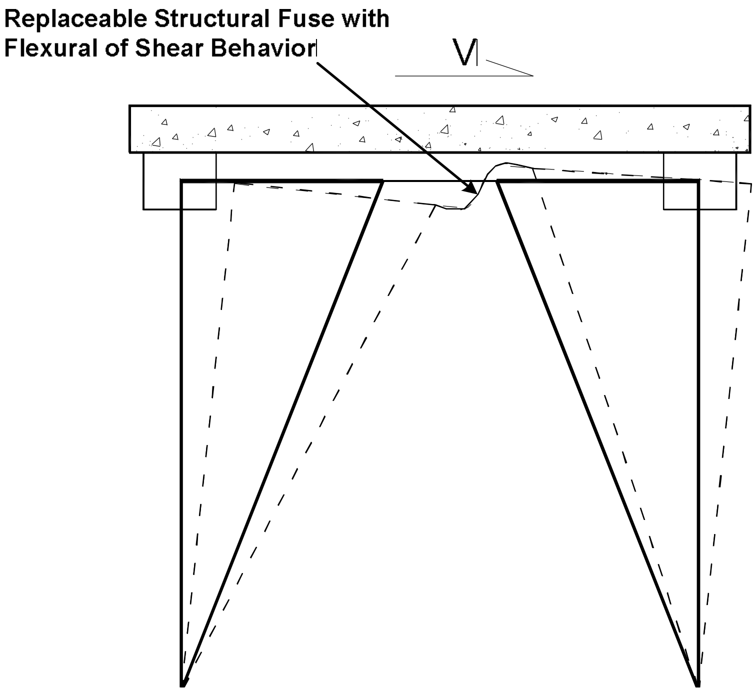


Figure 9. Schematic deformation pattern of the pier

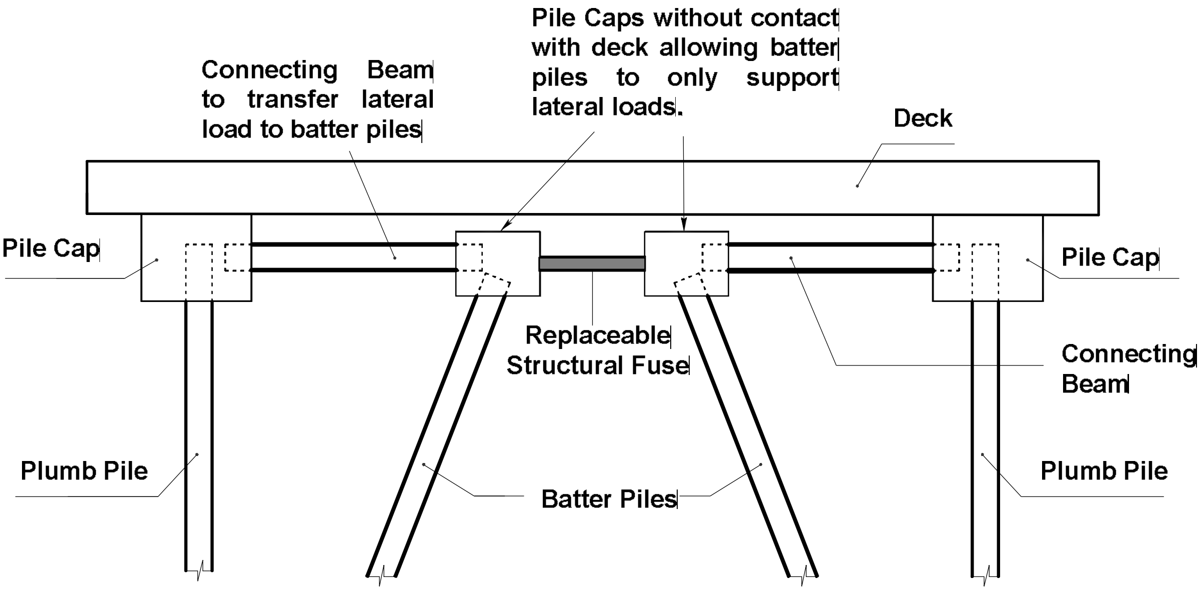


Figure 10. Proposed Detail for Combining Batter Pile and Plumb Pile

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