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SEISMIC RETROFIT OF THE SAN MATEO-HAYWARD BRIDGE

J. VITEK, R. VERMA, and M. R. BANAN

CARTER & BURGESS, 2500 Michelson Suite 100, Irvine, CA, 92715, USA

ABSTRACT

This paper briefly addresses the main issues related to the performance evaluation, vulnerability assessment, and retrofit design concepts of steel subsystems and foundation substructures of the San Mateo-Hayward Toll Bridge. After the description of the main structural subsystems of the bridge, the serviceability and overall performance criteria and design ground motions are described. The analytical models used to identify the vulnerabilities and confirm the retrofit schemes for the bridge are explained. Finally, the vulnerabilities and selected retrofit schemes are summarized for: (1) the superstructure steel box girders and their expansion hinges, (2) multicell steel towers and box spandrel beams for the steel piers, and (3) the rectangular and belled foundation substructures.

KEYWORDS

San Mateo-Hayward Bridge, Seismic Retrofit, Bridge Performance/Serviceability Criteria, Ductility Enhancement and Strengthening, Isolation and Energy Dissipation, Steel Bridge Vulnerabilities, Steel Box Girder, Multicell Steel Column, Precast Belled Pile Cap, Foundation Retrofit, Steel Pier Retrofit, Expansion Hinge Retrofit

1. INTRODUCTION

The State of California, through the Business Transportation & Housing Authority (BT&HA), has embarked on the task of seismically retrofitting major toll bridges in the Northern and Southern California locations. In order to meet a very tight construction schedule, the state has engaged consultants to provide retrofit strategies and PS&E construction packages for the six toll bridges. The Carter & Burgess Team was selected for the San Mateo-Hayward Bridge located in the San Francisco Bay area.
The objective of this paper is to highlight the key vulnerabilities associated with this bridge and describe the selected retrofit strategy for the same.

1.1 Location and Description of the Bridge

The San Mateo-Hayward Bridge was one of the first major bridgres orthotropic steel spans in the United States and was opened to traffic in 1967. The bridge carries six lanes of traffic on State Route 92 between Foster City and the City of Hayward and spans the south arm of San Francisco Bay. The total length of the bridge is about 7.1 miles and is comprised of three distinct sections: (1) West Approach-a 220 feet long concrete tee beam structure on two column bents, (2) Main Span Structure-a 36 span 1.85 mile long twin steel box girder with light weight concrete and orthotropic steel decks, and (3) Trestle Structure-a precast concrete trestle section composed of 859 spans each of 30 feet uniform length. The main span structure is the subject of this manuscript.

Substructure of the main span structure consists of 38 piers of which only Pier 1 is. on the land; the remaining 37 piers are in water. Pier 1 is a reinforced concrete frame building and houses the bridge paint shop and other maintenance and utility functions. Piers 2-11 and 30-37 are single and double level reinforced concrete portal frames on rectangular spread footings. Piers 12 and 13 are double level portal frames on rectangular spread footings. The upper level is composed of two cellular steel towers connected at the top by a cellular steel spandrel beam. The lower level is composed of two hollow reinforced concrete circular shafts connected at the top by a rectangular reinforced spandrel beam. Piers 14-18 and 21-27 are the same as Piers 12 and 13 but are supported on circular, bell shaped, pile supported foundations. The channel span Piers 19 and 20 are double level frames supported on circular, bell shaped, pile supported foundations. The upper level of Piers 19 and 20 is the same as their adjacent piers but the lower level is composed of four hollow, reinforced concrete, circular shafts arranged in a rectangular pattern and connected at the top by rectangular, reinforced concrete spandrel beams and a reinforced concrete slab. Piers 28 and 29 are similar to Piers 14-18 and 21-27 except that the upper level frame is composed of two reinforced concrete columns tied together at the top by reinforced concrete spandrel beams. Pier 38 is a reinforced concrete shear wall type building. The east wall of the Pier 38 supports the beginning span of the precast, 5.1 mile long, concrete trestle section.

Steel Superstructure comprises dual steel box girders of 37 spans. The spans generally consist of short spans (186', 206', 208') over the low rise sections, 292' spans over the intermediate rise sections, 375' spans either side of the navigation channel and a 750' main span over the navigation channel, which has a clearance of 138' to mean sea level. The structure is arranged in 18 single span anchor units alternating with 19 suspended spans. The suspended spans are supported by link plate hinges fabricated from T-1 steel. Alternating hinges are tied with pins and angle bars to prevent longitudinal movement. The short spans extend from Pier 1 to the fixed hinge at Pier 11 and from the fixed hinge at Pier 28 to Pier 38. These spans have a light weight concrete deck. The remaining spans have a steel orthotropic deck protected by an asphaltic concrete wearing surface. The suspended spans are supported by link plate hangers from cantilever sections at the anchor spans. These link plates constitute the hinges in the structural system. The hinges are restrained from transverse movement by shear locks located in the plane of the top and bottom flanges of the box girders. The steel box girders are supported on steel pin type bearing assemblies welded up out of plate and designed to permit rotation but prevent translation.

1.2 Previous Earthquake Damage

During the 1989 Loma Prieta earthquake, structural damage was found at the four bearing assemblies at the top of Piers 18 and 21. Due to relative movements of girders and columns perpendicular to the axis of the bridge, the 5-1/2 inch diameter pins connecting columns to the girders had moved in transverse direction. After the earthquake, the pins were out of position a maximum of 4 inches at Pier 21. The paint scratch marks on the pins indicated that transverse displacements of about 6 inches has occurred during the earthquake. The bearings were subsequently repaired by jacking the pins back into place and installing new pin retainer plates. Another area of damage was movements at the bases of steel box columns relative to the base plate at one column on each of the Piers 16 and 18. At location of Pier 19, the tie plate of fixed hinge of south box girder had ruptured. At location of Pier 38, the trestle units had moved relative to the pier in longitudinal and transverse direction causing the trestle units to come off its initial support and drop to the seismic retrofit brackets that existed at this location.

1.3 Bridge Performance/Serviceability Requirements

The seismic performance criteria for Toll Bridges in the San Francisco Bay Area are summarized as follows:

Table 1. Performance/Serviceability Requirements

Ground Motion	Minimal Performance Criteria	Important Bridge Performance Criteria
Functional Evaluation Earthquake (FEE)	(3) Immediate Service Repairable Damage	(4) Immediate Service Minimal Damage
Safety Evaluation Earthquake (SEE)	(1) Limited Service Significant Damage	(2) Immediate Service Repairable Damage

The preferred retrofit strategy for the San Mateo-Hayward bridge relates to the Criteria specified in block (1) of Table 1. The *Safety Evaluation Earthquake (SEE)* is based on the 84th percentile rock spectra from the deterministic event. For the major bridges in the San Francisco Bay Area, the rock spectra for the maximum credible event on the San Andreas fault system corresponds to the 1000-2000 year return period Uniform Hazard spectra. The *limited serviceability* is defined as the limited access (reduced lanes, light emergency traffic) possible within days and full service restorable within months.

Significant Damage: is defined as damage that can occur in the form of significant yielding, local buckling or local and limited fracture, shear failure of limited number of bolts in a connection but does not result in partial or full collapse during or after the earthquake. The significantly damaged structure should still have sufficient strength and stiffness to resist its own dead load and very limited live load (emergency and/or repair equipment traffic). Significant damage may require closure of the bridge for significant repair and replacement of components. Due to the difficulty of inspection and repair, significant damage in underwater portions of bridge shall be avoided.

1.4 Ground Motions

At the bridge site, soft to medium stiff clays (Bay Mud) are present at the Bay bottom and extend to depths up to a maximum of approximately 50 feet below the Bay bottom. Beneath most of the length of the high steel bridge, the Bay Mud soils have been largely removed by dredging. Beneath the Bay Mud, the soils consist of alternating layers of cohesionless soils (sands, sandy soils) and cohesive soils (clays, clayey silts) to the bottom of borings drilled for the bridge. The maximum depth of the existing boring is approximately 300 feet below the Bay bottom. These sediments overlay a rock formation which outcrop at Pier 1 and slopes down to the east at about 10 degrees.
Site-specific safety evaluation seismic ground motions were developed by means of site response analyses, using rock motions provided by Caltrans. These mudline motions consist of incoherent wavefields, representing the see event. In this case, the postulated SEE is a richter magnitude 8+ seismic event associated with the San Andreas fault system, which governs over the Hayward fault system for the steel bridge in terms of amplitude of the rock acceleration response spectra. The variation of the ground motions along the bridge alignment is quite significant, both in terms of intensity and frequency content.

2. ANALYTICAL MODELS

A comprehensive set of linear and nonlinear static, modal, and dynamic analyses were conducted to assess the seismic performance of the San Mateo-Hayward main span structure. The analytical models were developed at global, semi-global, local, and member levels and for soil-structure interaction. Global and semi-global models were developed to confirm vulnerabilities and to represent the proposed retrofit schemes. The semi-global models for four segments of the bridge were used to study the dynamic behavior of the bridge along its length and to determine maximum demand values. Additional local and member/component models were developed to evaluate capacities (strength and ductility), vulnerabilities, and performance of selected bridge sub-systems through a seismic event. Especially, member/component models were needed to confirm capacity of existing important members and components such as hollow concrete shafts, multi-cell steel towers, bell pile caps, expansion/tied hinges, H-piles, and pile groups that are not clearly represented or sometime not addressed in the state-of-the-art literature and codes.

2.1 Global MSTH Analysis

The global seismic response evaluation were carried out by the multi-support time history (MSTH) analysis method using the SADSAP computer program. Multiple-support input ground motions were specified at 11 locations (piers 1, 4, 9, 13, 16, 19, 20, 24, 28, 32, and 38) and were interpolated for the remaining piers. The mudline motion displacement time histories were used as the excitation for the analyses. The model consists of approximately 10,400 degrees-of-freedom (dof), and it is composed of an assemblage of 37 mathematical sub-structures (super-elements), and included the geometric non-linearities associated with all of the 18 expansion hinges, together with limited shear capacity fusing of spandrel beams and limited rocking of steel tower bases.

2.2 Member/Component Models

The global analyses that have guided retrofit strategy development have shown demands on many components that are well in excess of design strength capacities. However, since the performance criteria for the bridge allows for post-yield deformation, and since the behavior of the global model must be ascertained for these levels of component deformation, studies of critical component behavior were performed. The stiffness and nonlinear behavior of certain critical components were modeled in detail to (1) improve the characterization of components in the global model and (2) to calculate capacities and limit state behavior.

The steel components analyzed were modeled using the ANACAP constitutive modeling package in conjunction with the ABAQUS general purpose finite element program or the DRAIN-3DX program. The low cycle fatigue law embedded in the model was used to guide judgment on assessing the number of cycles to failure for a particular strain condition. The concrete component/member models were all analyzed using the ANACAP constitutive models. Within the concrete constitutive model, cracking and all other forms of complex nonlinearity are treated at the element level. The reinforcement in the continuum models was modeled as individual sub-elements within the 3D concrete continuum elements. The rebar sub-element stiffnesses are superimposed onto the concrete element stiffness in which the bar resides. The rebar material behavior is handled with a separate constitutive model that treats the steel plasticity, strain hardening, and bond-slip behavior.

3. SEISMIC RETROFIT ALTERNATIVES

The two Global Retrofit Strategies were considered for the main span structure of the San Mateo-Hayward Toll Bridge and are summarized below:

3.1 Global Retrofit Strategy I- Ductility Enhancement and Strengthening

This retrofit strategy approach incorporates the conventional bridge seismic retrofit combination of strength and ductility enhancement. Hence, the overall retrofit approach is to minimize the D/C ratios of the various bridge components under SEE to certain acceptable criteria. It should be noted that the conventional retrofit methods selected in this strategy will not only lower the overall D/C ratios for internal forces and displacements, but also assist in an essential stable and ductile hysteretic performance during a major seismic event such as the SEE at the bridge site.

3.2 Global Retrofit Strategy II-Isolation, Damping and Energy Dissipation

This retrofit strategy approach incorporates the concept of providing a hybrid system of isolation and damping in order to tune the structural response of the bridge to the required performance criteria and typically reduce the seismic forces being input into substructure and superstructure. The isolation bearings usually provide a period shift for the bridge in the longer period range for a reduced seismic response, together with the added benefits for a potential elastic or limited ductility response of the sub-structure. In addition, the damping devices at bearing locations and expansion hinges work towards reducing the impact velocity and the overall global and component drift levels to certain acceptable criteria.

The isolation-based retrofit strategy was not selected as the preferred strategy for the San Mateo-Hayward bridge because:

- Total retrofit cost is higher than the Strategy I.
- There is no major problem with the superstructure, therefore no significant cost-reduction is achieved for isolating the superstructure.
- Longitudinal drifts for towers and shafts do not reduce significantly with isolation, since: (a) in average, 70% of the mass is accumulated in the piers and 30% in the deck, (b) piers behave independently in the longitudinal direction, and (c) top of the piers have no lateral restraint.
- Longitudinal periods of different segments of the bridge are large (2.8 -3.8 sec). Therefore, no significant reduction of forces is achieved due to the period elongation which is customarily associated with seismic isolation.
- Tower/shaft base locations for isolation would seem to be the structural location for the devices. However, the constructibility and cost issues were prohibitive for this location.

4. BRIDGE SUBSYSTEMS: VULNERABILITIES AND RETROFIT SCHEMES

The preferred retrofit strategy for the main span bridge structure relates to a ductility enhancement and strengthening approach. The following sections describe this retrofit concept as it relates primarily to the steel portion of the bridge and the foundations.

4.1 Steel Superstructure

Steel Box Girders have vertical capacities limited by local buckling or yielding capacity. Although the box girders form relatively rigid units between the deck hinges and the piers, the existence of fatigue sensitive details, especially on fracture critical components, requires that seismic loading of the box girders be limited and stresses at fatigue sensitive locations be checked. For loadings in the direction of the design loads the susceptibility to local buckling is low. In the transverse direction, the initial capacity limit state considered is based on maximum seismic stresses well below the elastic limit. In general, the seismic demands on the box girders are expected to be lower after a retrofit scheme is implemented.

Proposed Retrofit for Steel Box Girders: Do nothing (unless dictated by construction loading during bearing retrofit).

Pin-Hanger Assemblies at the deck hinges are critical links within the seismic load transfer path from the deck to the substructure. They contain fracture critical elements and fatigue sensitive details, and their loading and behavior during an earthquake is quite uncertain. The deck hinges are also required to accommodate large seismic displacement demands. Failure of the pin-hanger assemblies can result in span loss. Due to a combination of high stress concentrations, thick plates, possible flaws in pin plates caused by slotting of pin holes during erection, rapid loading and high stresses due to unintended fixity at the pin, failure due to fracture may govern. Redundancy is an important criteria for the pin-hanger assemblies. Within a pin-hanger assembly on the 208 and the 292 foot spans failure of a pin plate, a pin or a hanger bar can result in failure of the whole assembly. There is only one pin plate per joint, and if one hanger bar fails, the pin could rotate and allow the other hanger bar to come off. Therefore, retrofit to increase redundancy is recommended. On the channel span there are two pin plates per joint, and therefore, if one eyebar fails the assembly will remain stable. Although there is some potential for pin instability if one pin plate fails, retrofit of the pin hanger assemblies on the channel span is not recommended at present, pending results from a more detailed analysis.

The existing hinge ties are quite vulnerable to seismic loading, but if a system that can accommodate the relative span movement demands is provided, the consequences of hinge tie failure are minimal. Failure of the hinge ties could even improve the overall seismic response of the bridge by allowing independent longitudinal response of the deck units and a balanced load distribution to all piers. The existing hinge ties may be modified so that they could act as longitudinal fuses during a seismic event.

The existing hinge shear locks have limited strength and displacement capacity. Failure of the shear locks is undesirable and can lead to significant damage in the pin-hanger assemblies and the box girders.

***Proposed Retrofit for Expansion and Tied Hinges**:* Two pin hanger assemblies at each deck hinge (except piers 19 and 20) need to be added together with replacement of top and bottom shear locks (see Fig. 1).

4.2 Steel Towers, Spandrel Beams and Tower Anchorage Assemblies

Steel Towers consist of multi-cell, welded, steel box columns. All towers are connected with a single cell, rectangular, ***steel spandrel beam*** The developed member models for steel towers were capable of capturing overall and local buckling phenomena and represented reasonable patterns of local buckling. From the preliminary analyses, it was observed that the as-built steel towers are vulnerable to local and overall buckling and compression yielding. With increasing the height of steel towers, their vulnerability to overall buckling increases and to local buckling at tower leg decreases. The existing towers do not have sufficient ductility capacities to accommodate the large drifts caused by the seismic event. The steel spandrel beams are vulnerable to shear buckling.

Proposed Retrofit for Steel Towers: (a) Additional ASTM A36 steel plates need to be added to the outside of tower columns-plate thickness ranges from 3/8" to 5/8" and (b) Additional longitudinal and transverse stiffeners (W 6x15 typical) are needed (see Fig. 4).

***Proposed Retrofit for Steel Spandrel Beams**:* (a) Additional ASTM A36 steel plates are needed inside the spandrel beam-plate thickness ranges from 1/2" to 7/8", resulting in 4 to 6 cells within the original section and (b) Transverse stiffeners inside the spandrel beam (W 6x15 typical) are needed (see Fig. 3).

Steel Tower Anchorage Assemblies affect the ductile performance of the steel towers needed to accommodate the large drifts produced by the seismic event. Non-ductile failure modes should be removed for these components. The tower base assemblies are vulnerable to shear failure of the anchorage bolts as well as yielding of the anchorage bolts. Any strengthening of the tower base assemblies for the shear failure mechanism will require a strengthened connection between the tower and the baseplate to transfer the shear forces, since the towers were not connected to the baseplate for the original construction contract.

Proposed Retrofit for Steel Tower Anchorage Assemblies: (a) Splice ductile steel bolts to existing anchorage bolts above the base plate to ensure that yielding of the embedded anchorage bolts is minimized, (b) Add shear keys through base plate into concrete piers, and (c) Add ductile steel angles connecting tower base to the base plate to improve system ductility for rocking under cyclic loading (see Fig. 2).

4.3 Foundation Substructure

Rectangular footings range from 68'-0" to 72'-0" in length; 20'-0" to 24'-0" in width; and 5'-0" to 6'-0" in depth. The existing piles are HP14 x 89 and are embedded 6" into the foundation. All perimeter piles are battered. The number of the existing piles vary from 53 to 84 at different piers. The belled concrete pile caps serve to transfer loads from the column shafts to the steel piles. The belled caps are constructed with precast concrete shells and concrete tremie into which the piles are embedded. The H-piles are embedded a minimum of 12 feet into the tremie concrete except for the plumb piles in the outer ring which are only embedded 9 feet.

The vulnerabilities of the rectangular foundations are: (a) Drifts in excess of 6 inches fail the battered piles and significant number of the plumb piles, (b) Piles posses insufficient tensile capacity for uplift due to their minimal development of 6" into the footing tremie, and (c) Rectangular pile groups for two-level piers are vulnerable to foundation overturning in the longitudinal direction. The vulnerabilities of the bell foundations are: (a) Battered and plumb piles are vulnerable to local buckling due to excessive drift. Under these drifts , the battered piles will likely lose their bearing capacity. The remaining plumb piles will not resist the seismic demand moments., and (b) The belled pile cap has inadequate confinement in the bell shells.

***Proposed Retrofit for Rectangular Foundations**: (a)* Provide a 30" minimum footing overlay, (b) Add upper and lower mat of reinforcement in the new concrete, (c) Add drill and bond dowels into existing concrete footing, (d) Add additional shear reinforcement, and (e) Add four large diameter steel pipe piles filled with concrete at the corners of each footing (see Fig. 5).

Proposed Retrofit for Bell Foundations: (a) Add hoop reinforcement around bell shell and tremie, (b) Add two large diameter steel pipe piles filled with concrete per pier (except piers 19 & 20), and (c) Confine bell shells, concrete tremies, and drilled shafts by steel casing filled with tremie concrete (see Fig. 6).

5. CONCLUSIONS

The San Mateo-Hayward toll bridge, as a major San Francisco bay crossing, represents significant challanges in terms of seismic vulnerability assessment and retrofit strategy development. These challenges relate to the long span and curved geometry of the bridge, variation of structural configuration of the bridge spans and the substructure, geometric and material non-linearities, soil-structure interaction, constructibility, allocated cost and traffic handling issues. The preferred strategy was selected on a fast-track basis incorporating existing design guidelines and proven retrofit measures, together with the development of new criteria and several innovative solutions as applicable to some of the atypical details of the bridge, such as multi-cellular steel towers, hollow RC shafts and RC bell foundations.

The previous studies conducted on this project have been extended into alternate retrofit strategies and cost estimates for use by the State in selecting the final retrofit strategy. The final retrofit strategy has now been selected. PS&E documents are being prepared incorporating the selected final strategy.

6. ACKNOWLEDGMENTS

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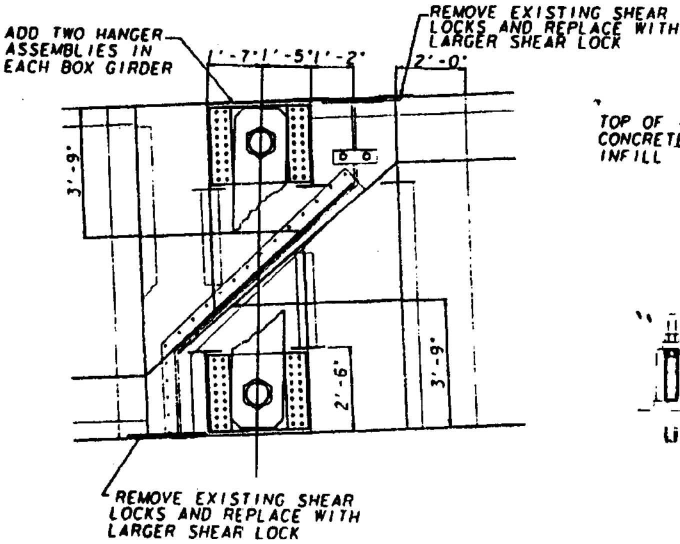


FIG 1. EXPANSION HINGE RETROFIT

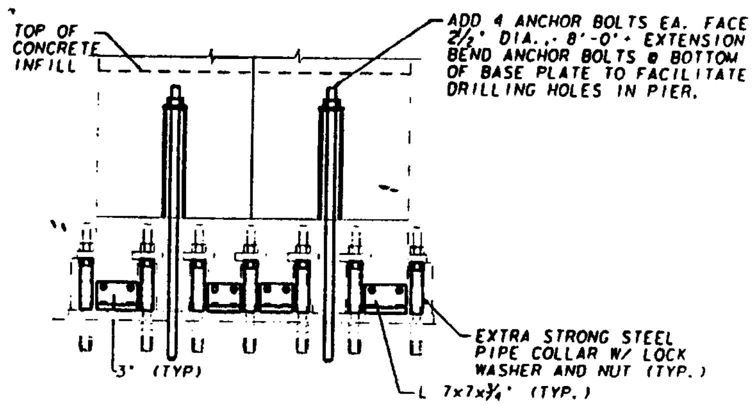


FIG 2. STEEL COLUMN-BASE RETROFIT

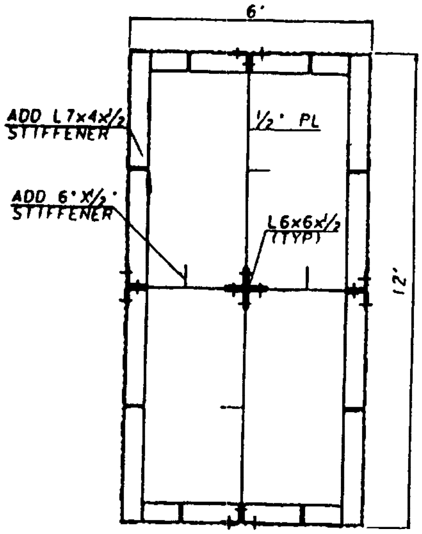


FIG 3. STEEL SPANDREL BEAM RETROFIT

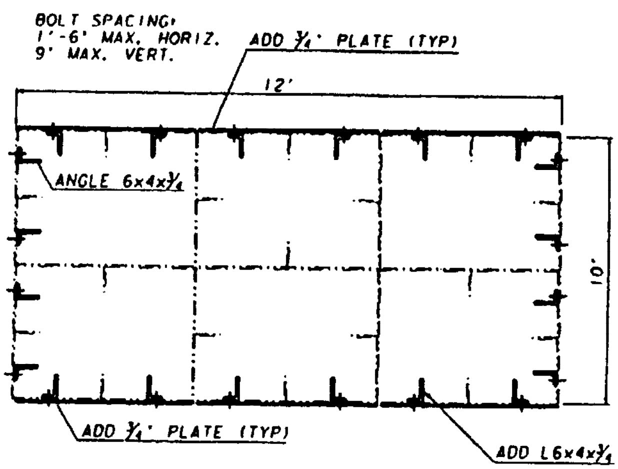


FIG 4. STEEL MULTI-CELL TOWER RETROFIT

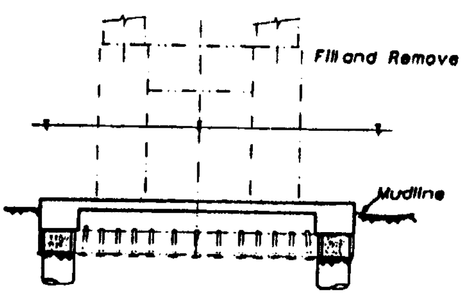
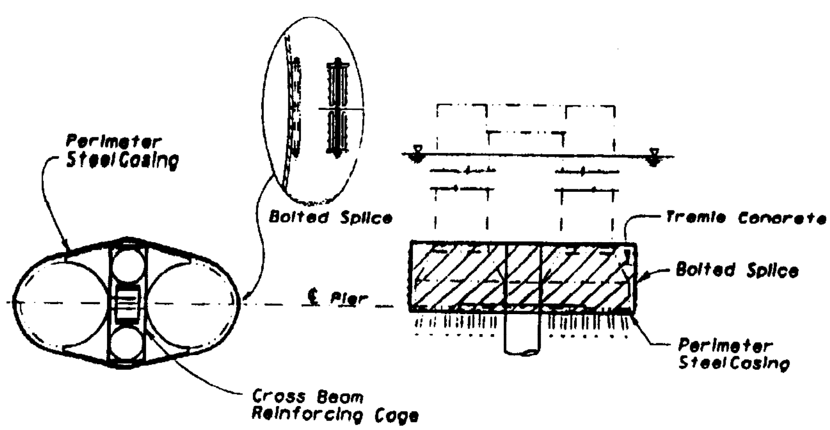


FIG 5. RECTANGULAR FOUNDATION RETROFIT



(A) PILE CAP PLAN

(B) ELEVATION

FIG 6. BELL FOUNDATION RETROFIT

LEGEND : —··· AS-BUILT
—— RETROFIT