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Seismic Retrofit of the North Approach Viaduct of the Golden Gate Bridge

Consolidation parasismique du viaduc nord du pont de Golden Gate

Seismische Nachrüstung des North Approach Viaduct der Golden-Gate-Brücke

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SUMMARY

This paper describes the seismic retrofit of the North Approach Viaduct of the Golden Gate Bridge. A three-dimensional model of the complete viaduct was developed. Both the response spectrum analysis using 100 modes and time history analysis were carried out for the as-built and retrofitted structure. Three earthquake input components were considered. The retrofit schemes included replacing the existing towers with new towers, replacing steel bearings with isolation bearings, replacing the existing footings with new pedestals and footings with tiedowns, providing an expansion joint device at the Pylon and North Abutment, and replacing and strengthening superstructure bracing members.

RÉSUMÉ

Cet article décrit la rénovation parasismique du viaduc nord du pont de Golden Gate. Un modèle tridimensionnel du viaduc complet a été établi. Trois cas de séismes ont été considérés. Les projets de rénovation comprenaient le remplacement des tours existantes par de nouvelles tours, le remplacement des appuis d'acier par des appuis isolants, le remplacement des plaques de base par de nouveaux socles et fondations avec ancrage, l'introduction d'un assemblage à joint de dilatation au pylône N2 et à la culée nord, et le remplacement ou le renforcement d'éléments de la superstructure.

ZUSAMMENFASSUNG

In diesem Artikel wird die seismische Nachrüstung des North Approach Viaduct der Golden-Gate-Brücke behandelt. Ein dreidimensionales Modell des gesamten Viadukts wurde erstellt. Die Nachrüstungspläne beinhalteten das Auswechseln der vorhandenen Pylonen gegen neue, Austausch der Stahlauflagerungen gegen Isolierungsauflagerungen, Auswechseln der vorhandenen Fundamente gegen neue Sockel und verankerte Fundamente, Erstellung eines Dehnungsfugensystems am Pylon N2 und North Abutment (nördliches Widerlager) sowie den Austausch und die Verstärkung der Oberbauverstrebungsteile.

1. DESCRIPTION OF THE EXISTING NORTH APPROACH VIADUCT

The North Approach Viaduct of the Golden Gate Bridge extends from the north end of the North Anchorage Housing to the North Abutment. It consists of five truss spans of 53.3m each on partially straight and horizontally curved alignment. These truss spans are supported on four intermediate steel braced frames, on Pylon N2 on the south end, and on the North Abutment. The braced frame tower heights are about 15.2 m at Tower 1, 39.6 m at Tower 2, and 45.7 m for Towers 3 and 4. The original construction of the viaduct consists of two parallel trusses spaced 15.2 m apart. Each braced frame tower has 4 vertical legs and plan dimensions of 15.2 by 15.2 m. As part of a 1961 widening project, an additional line of trusses and two additional vertical legs at each tower were added. These new members are located 25 feet west of the original structure. The towers are supported on concrete pedestals and spread footings at the original bridge and caissons at the widening.

In 1982, the viaduct was retrofitted to the then-current Caltrans seismic retrofit guidelines. The following details were used: 1) threaded bar restrainers oriented in the longitudinal direction were installed at bearing supports to prevent excessive relative displacement; 2) transverse restrainers were installed to tie the truss spans to the supporting towers; 3) additional diagonal bracing members were added to the widening portion of the towers and at the lower portion of the original towers; and 4) foundations at the tower legs of the widening section were strengthened to provide stronger anchorage. During the early 1980s, the original concrete deck was replaced with a lightweight orthotropic steel deck system.

2. ANALYTICAL MODEL AND ANALYSIS METHOD

2.1 General

The seismic evaluation and retrofit strategy determination were based on the linear response spectrum method, the linear elastic time history analysis method and the nonlinear time history analysis method. The computer program IAI-NEABS, which is an enhanced version of NEABS (Nonlinear Earthquake Analysis of Bridge Systems), was used for these studies[1].

2.2 Linear Elastic Analysis

A three dimensional model of the complete viaduct was developed to perform a linear elastic time history analysis and linear elastic response spectrum analysis. Several models were used in the linear elastic response spectrum analysis and the linear elastic time history analysis of the as-built structure. The vulnerable components were identified as follows:

1. Restrainers,
2. Abutment connections,
3. Foundations (anchor bolt uplifting),
4. Bearings (rocking response of the tall bearings), and
5. Tower braces and column members.

The behavior of these components were modeled as nonlinear and/or inelastic elements in subsequent runs.

Various retrofit schemes were modeled using equivalent linearized parameters for various components included. The analyses were carried out either by the linear time history analysis method or linear elastic response spectrum analysis method.

2.3 Nonlinear Analysis

A three dimensional nonlinear model of the complete viaduct was developed. The nonlinear time history analysis was performed using the step-by-step direct integration of the coupled equations of motion. At the end of each time step, the equilibrium condition was checked. If the maximum unbalanced residual force was too high, the program automatically reduced the integration time increment by subdivision. Within each subdivision step, further equilibrium iterations were performed to assure accuracy of the solution.

The nonlinear process consisted of an iterative or "piece meal" process using the initial linear analysis and subsequent nonlinear analyses to establish the basis for the model refinement in the next level of the nonlinear model and analysis (i.e., identifying members that should be modeled as nonlinear in the next run). This process allowed for an early retrieval of data and a review of the structural responses of the adjusted model for each run. With a clear understanding of the limitation of each model and the resulting responses, engineering judgments were made regarding the adequacy of the "partial nonlinear" model for the intended purpose of the analysis (i.e., to identify the vulnerability and the potential retrofit scheme).

As the process of model refinement continued, the effect on the structural responses caused by the modification of the model gradually decreased. Eventually, this iterative nonlinear analysis was stopped when either all members were modeled as nonlinear or no additional nonlinear response were expected.

3. SEISMIC DEFICIENCIES

The following is a description of deficiencies in various components throughout the structure.

3.1 Superstructure

Many truss members need to be reinforced or replaced to meet strength (kl)/r and b/t ratio requirements. The capacity of existing longitudinal and transverse restrainers must be increased. Existing deck joints are inadequate for the anticipated lateral displacements. Truss longitudinal connections to Pylon N2 and North Abutment are inadequate.

3.2 Bearings

Existing steel bearing anchor bolts fail in tension creating possible instability and unacceptable vertical displacements. Bearing support widths are inadequate to ensure that trusses will be supported after the earthquake. Transverse restraint is inadequate to handle forces and displacements.

3.3 Towers

Most connections of diagonal braces and horizontal struts are inadequate to develop the computed tensile forces or the yield strength of the member. In many diagonal bracing members the allowable demand/capacity ratios are exceeded. Column elements have a demand/capacity ratio higher than 1. Anchor bolts that connect the tower base to the concrete pedestals will fail in tension and shear.

3.4 Foundations

Concrete pedestals are underreinforced and overloaded and will fail in a brittle manner. The footings provide insignificant resistance to uplift.

3.5 North Abutment and Pylon N2

The North Abutment is inadequate to absorb lateral forces and displacements. The superstructure will impact the North Anchorage Housing during lateral motions at Pylon N2 and existing longitudinal and transverse restrainers are inadequate.

4. SEISMIC RETROFIT DETAILS

Brief descriptions of the retrofit procedures are discussed below.

4.1 Temporary Supports

Temporary supports will be required at each tower to support the structure while bearings and towers are replaced. They will be designed so that the temporary towers can be reused at all four towers. New temporary foundations and strengthening of the superstructure truss will be required.

4.2 Isolation Bearings

Isolation bearings will be installed at the top of the new towers. Bearing performance will be confirmed by a comprehensive series of tests which will be performed over the life of the structure so that the effect of aging will be known. Provisions will be made for bearing replacement should that be needed. Inspection of the bearings will be straightforward, only routine visual inspection need be performed. In particular, inspection of the bearing outer cover to verify that it is free of large cracks, tears, etc. would be performed. The retrofit design will make some provision for replacement of the isolation bearings. The new jacking points were built into the bearing assembly as a part of the retrofit. The new isolation bearings at the top of the towers will serve to dissipate horizontal loads but there are also significant vertical uplift loads which must be transmitted from the superstructure to the tower. Vertical restrainers will be incorporated into the design.

4.3 Tower Foundations

The concrete foundations under each leg of the towers have minimal reinforcement. Originally it was proposed to encase each pedestal on all four sides with a properly reinforced and tied collar; however, it was found that replacement of the pedestals and footings was more cost effective. Also, because the seismic forces are generating large overturning moments, tension tiedowns are required at each tower footing.

These tower overturning moments generate high overstresses and hence large elongations in the existing anchor bolts, which connect the tower legs to the concrete foundation. Additional anchor bolts and shear pins will be required.

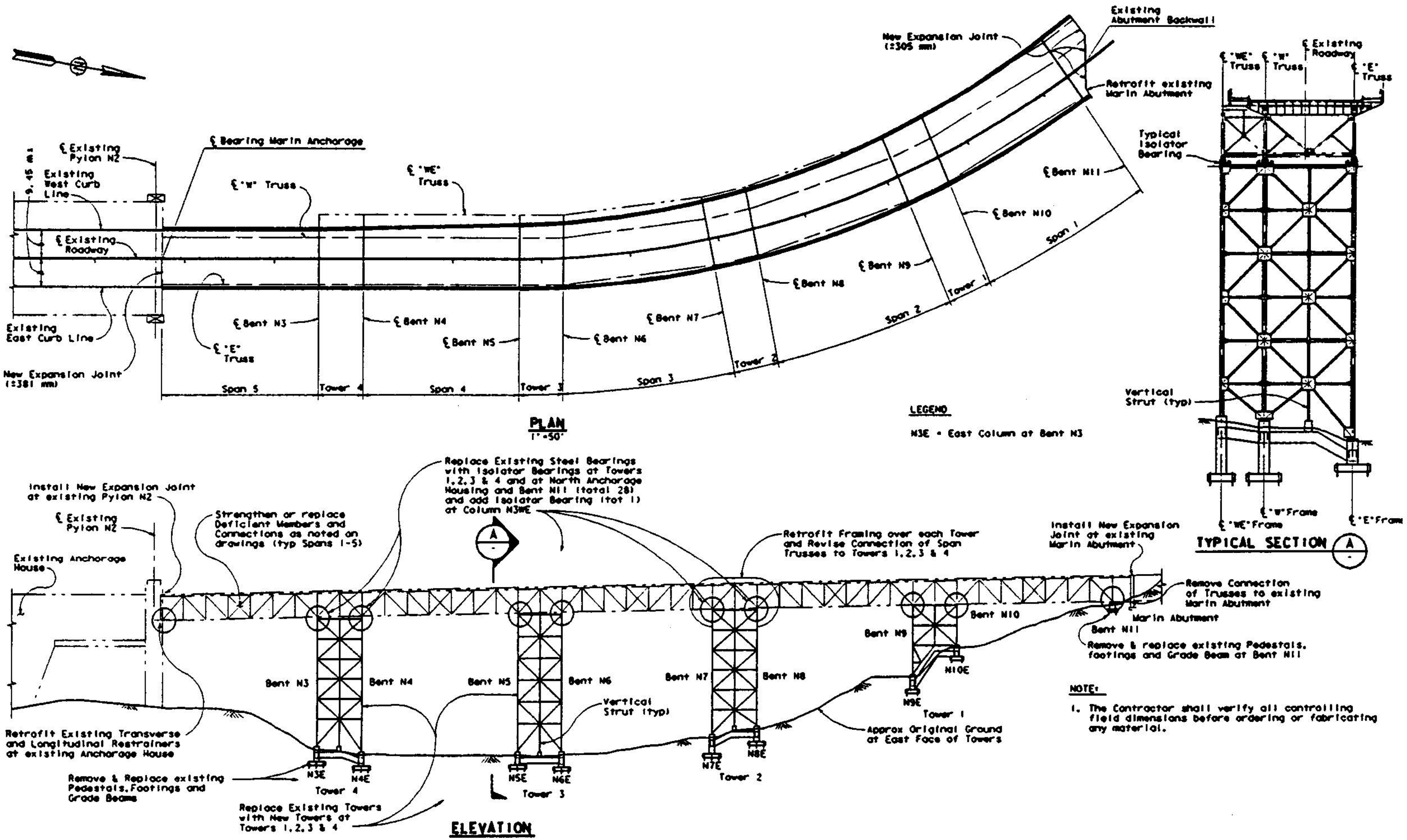

Figure 1 Seismic Retrofit of the North Approach Viaduct

4.4 Superstructure

The lateral seismic loads overstress both the top and bottom lateral bracing systems and if transferred to the main truss members they will be overstressed and require strengthening. The entire bottom lateral system in the tower box, immediately above the towers, will be strengthened to accommodate the transfer of loads between the superstructure and substructure. Other bottom lateral members will be replaced, added, or strengthened. Several top lateral bracing members must be strengthened, added, or replaced. The main truss verticals located directly above the temporary support towers will be strengthened to handle the tower reactions which are temporarily brought into the truss. Also, other vertical truss members in the spans need retrofitting. The sway bracing in the widening and main superstructure will be strengthened and retrofitted.

4.5 End Supports (North Abutment and Pylon N2)

At the North Abutment and Pylon N2 there will be large longitudinal and transverse displacements which cannot be accommodated by the existing joints. Therefore, a large movement joint will be installed.

4.6 Towers

The towers will be replaced; however, had we decided to retrofit the existing towers, the following work would have been needed:

- Rivets at Joints
  Some of the rivets in the joints of the tower members would have to be replaced with high strength bolts.

- Tower Diagonals
  Some diagonal members would have to be strengthened to keep within the design criteria and some diagonals added in the widened portion of the tower.

- Tower Columns
  Column strengthening would be required to maintain a D/C ratio of 1.

- Top of Tower Lateral Bracing
  The horizontal bracing system at the top of the towers would be strengthened to make the top level of the towers as stiff as possible to transfer seismic forces from the superstructure to the towers.

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