From Research ... to Practice

Seminar Proceedings

SEISMIC DESIGN AND RETROFIT OF BRIDGES

Presented by:

University of California at Berkeley
Department of Civil Engineering
Earthquake Engineering Research Center

and

Califomiabepartrnent of Transportation
Division of Structures

held at

Physical Sciences Lecture Hall
University of California at Berkeley

[une 8 and 9,1992

Evaluation of the Seismic Performance of Retrofit Concepts for Double Deck Reinforced Concrete Viaducts

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ABSTRACT: Some of lhe issues introduced in lhe retrofit of double deck reinforced concrete viaducts are described in lhis paper along wilh preliminary results of experimental and analyúcal investigations focusing on lhe ability of suggested retrofit concepts to achieve performance goals. The fírst pan of lhe paper describes a one-lhird scale model of a portion of a retrofitted double deck viaduct lhat is being tested in lhe Berkeley Structures' Research Laboratory. The framing system for the retrofitted viaduct consists of new spiral reinforced columns, and in lhe longitudinal direction, new haunched edge girders and, in lhe transverse direction, partially post-tensioned bent caps. In lhe second portion of the papero analyúcal studies are presented to evaluate some basic assumptions uúlized in the retrofit process. These analyses ilIustrate the need for conservatisrn in selecting design forces as well as in detailing critical regions. Analyses presented inelude the elastic and inelastíc, static and dynamic performance of a double deck viaduct considering possible variations in the intensity of uni-directional as well as of bi-directional ground motions.

INTRODUCTION

As described in Reference 1, the potential seismic deficiencies encountered in existing double deck viaducts are numerous. These range frorn the inadequate strength andoespecially, ductility of columns, bent caps, and joints, to the complex and often inadequate lateral framing systems uúlized in lhe longitudinal direction of lhe roadway, and 10 irregular and highly unsymmetric framing systems. Because of lhe complexity of the viaducts, the need to simply design analyses, and the adverse consequences of creep and shrinkage in the post-tensioned girders often used in the upper levels, pin connections were often introduced in lhe existing double deck viaducts at lhe ends of many columns in order 10 make lhem statically determinant. This lack of redundancy is also viewed as a serious seismic deficiency.

Field tests of a retrofit portien of the 1-880 Cypress Street viaduct in Oakland following the Loma Prieta earthquake indicate [2] that a variety of techniques may be used to strengthen a viaduct in the transverse directíon to a level consistent wilh Iife safety. However, lhe studies indicate lhat lhe performance of the various retrofit details are no! well understood and damage obtained may not allow conúnued operation of the viaduct, or even repair, following a major earthquake. Because double deck viaducts are important lifeline components, Caltrans has established a policy that the double deck viaducts in San Francisco should be able to be quickly repaired to remain operable following a design level earthquake.

The Cypress Viaductstudies also elearly indicated that it was necessary to view lhe structure as a system, and that piecemeal retrofit of individual elements was simply likely to transfer and concentrate damage in other elements that had not been retrofit, These earlier studies did not address any of the important issues related to the seismic resistance of the viaducts in the longitudinal direction.

Because of the unique problems associated with the retrofit of double deck viaducts and their important role as major lifeline components, a research investigation was initiated at Berkeley to assess the various technical issues needed to develop and validate design and retrofit guidelines. In this paper, the basic problems encountered in designing reliable retrofits are described and a test program aimed at validating and improving current retrofit schemes is presented. Analytical results are described to illustrate sorne of the important special issues related to the inelastic dynamic behavior of these systems.

BASIC RETROFIT STRATEGY VTILIZED FOR DOVBLE DECK VIADVCTS

In the retroñt of the remaining double deck viaducts in the San Francisco Bay Area a number of retrofit strategies have been developed by the design consultants. These evolved through a review process between the consultants. Caltrans, a Peer Review Panel and technical consultants. This review process resulted in a consistent design rnethodology, incorporating similar design assumptions and details, for the various double deck design projects in the San Francisco Bay Area.

The retrofits were generally based on current Caltrans design requirements [3] for new construction. That ís, seismic demands were cornputed using elastic analysis melhods based on a design response spectrum corresponding to a maximum credible earthquake in the vicinity of the viaduct. Standard Caltrans response spectra corresponding to appropriate local soil conditions and 5% of critical viscous damping were used in most cases. though site specific spectra were developed for soft bay mud sites. Because of various technical issues discussed below, and the desire to limit damage in these lifeline structures, the factor (Z) by which the elastic demands for the maximum credible earthquake could exceed the nominal capacity of elements was limited to four for standard bents. (Srnaller Z values were required for C-bents and for plastic hinges occurring at splices between new and existing construction.) Current Caltrans' provisions for design of new multiple column viaducts would permit Z values up to eight, depending on the period [3].

Current Caltrans' requirements stipulate that members, joints and connections away from the plastic hinge should be proportioned considering the plastic capacity of the hinge regions. For new construction [3] the plastic capacity may be taken as 1.3 times the nominal ultimate capacity of the critical section. For the retrofit structures, this ratio was used to define plastic strength when the entire critical load resisting systern was removed and replaced, but was increased to 1.5 in the case where retrofits consisted of strengthening existing sections by steel or concrete jacketing, or where only parts of the lateral load resisting system was replaced.

Current Caltrans' requirements also stípulate that bi-directional ground motion effects be considered. This is accomplished by detailing the structure to sustain 100% of the required lateral displacement in one direction while 30% of lhe required displacement in the orthogonal direction is simultaneously imposed. In the primary direction, the structure would yield and internal forces would be limited by the plastic capacity of the critical sections expected to yield. In the orthogonal direction, the yield capacity of the structure could be as low as 25% of the elastic demand (i.e., the member forces could be reduced by I/Z or 114 to obtain the required design force). Since 30% of the maximum elastic displacement in a direction would then likely exceed the yield displacement in that direction, members and joints need to be designed considering simultaneous yielding of the structure in both principal directions, as well as the standard case of yielding in each principal direction independently.

Structural System Used **in** Retrofits -- Because of the need to substantially strengthen the viaducts, and the problems associated with strengthening and confining the existing joint regions, the basic retrofit concept developed consisted of shoring the existing roadway, removing existing columns and joints and replacing them using construction consistent with modem seismic-resistant design practices, and strengthening the bent caps in the transverse direction using post-tensioning, mild reinforcernent, or a combination of these two. Various practical, economic and legal requirements necessitated the leaving the existing roadway relatively undisturbed.

Because of the difficulties in substantially strengthening foundations and of incorporating the upper deck level's post-tensioned caps into the lateral load carrying system, pins were generally introduced at the tops of columns in the upper level and at the bottoms of the columns in the lower level, The resulting H-shaped bent in the transverse direction is not highly redundant so that a high level of conservatism was warranted, and implemented in design and inspection.

In the longitudinal direction, the existing lateral load carrying system consists of the box girder system used to support the roadway connected eccentrically to the columns. Longitudinal framing action required transfer of moments from the columns through the joint to the bent cap. The bent cap would then acting in torsion transfer the moments lo the stems of the box girder. This resulted in two major problems. First, the box girder was not designed to resist the seismic loads required of the retrofit; substantial modifications to the box girder would be required, unless an altemative load resisting system could be developed. Second, the lateral load transfer mechanism from the column to the bent cap though torsion (or torsional shear friction) was difficult to proportion for the loads required and its inelastic performance was viewed as undependable without substantiation.

These difficuities in the longitudinal direction were addressed considering a variety of altemative load carrying systems. In sorne cases. steel braced frames were suggested. While these were technically feasible, they were abandoned due to problems in partially blocking access to the underside of the viaductothe often high uplift forces developed in the foundations. the apparent low ductility of the systems and aesthetic objections. The retrofit design evolved to the addition of longitudinal girders along the edges of the roadway framing between the centers of the columns. This provided a conventional moment frame to resist lateralloads in the longitudinal direction.

Two different basic details were developed for the edge girders. In one, the girders were separated from the roadway. This permitted, in principIe at least, the central portions of the girders to be precast and dropped in position, and the straight girders minimized the construction effort needed ro accommodate curving roadways. The isolated edge girder would also eliminate uncertainties in intemal forces that might occur in service due to creep and shrinkage in tbe new concrete. The transfer of the large inertial forces from the roadway to the exterior frarne, however, would be done through a combination of shear (friction) and bending in the short segment of bent cap between the column and the roadway. The second option cast the edge girder directly againsl the existing box girder. This increased the effectiveness of the edge girder, minimized problems in transferring inertialloads from the box girder to the longitudinal frame, and eliminated alignment problems where traffic could ron over the joint between the existing deck and the new edge girder. Construction of the integral edge girder where the columns are located away from the edge of the roadway (i.e.• in an outrigger configuration) or where the roadway is highly curved results in considerable added mass.

Expected Plastic Collapse Mechanism. -- Because of the high dead load moments in the caps. the caps are generally quite strong and it is difficult to develop plastic hinges in them under transverse lateral loading. Moreover, the need for a construction joint and splicing of reinforcement at the junction of the existing bent cap and the new colurnn, make lhe reliability of a plastic hinge at this interface questionable.

In the longitudinal direction, the fragility of the box girders supporting the roadway, the uncenainties regarding the torsion transfer capability of the bent cap (for lhe isolated edge girder), and the desire to minimize earthquake-induced damage in the roadway, resulted in the edge girders being designed to be stiff and strong. This would reduce relative rotations along the bent cap between the column and lhe box girder. It would also concentrate yielding in lhe columns, ralher lhan in the edge girders.

Column yielding is consistent wilh current Caltrans' practices for single level over crossings and viaducts. Various guidelines currentiy exist [3,4] for lhe proportioning and detailing such structural elernents, and for estimating the inelastic deformation capacities [5].of the columns and bent caps in such structures, However, double deck viaducts pose a number of new problems lhat must be resolved.

Weak Story Response. -- For example, it will be difficuit (and practically impossible) to insure yielding in both upper and lower columns. While the columns may be proportioned according to the maximum design forces computed (by an elastic dynamic analysis) at each level, the maximum forces at different levels will not generally occur at the same time because of higher mode effects, Similarly, prediction of the forces in lhe columns will be complicated by the bi-directional nature of the behavior and differences between elastic and inelastic response. As a result, it would be expected that yielding will concenLrate in one leve!. The distribution of deformations in the structure may then differ significantly from lhe elastic distribution. Estimation of ductility demands may not be a simple matter.

This situation is schematically illustrated in Fig. 1. For this simple case. each level is assumed to have lhe same elasto-perfectly plastic characteristics (the beam might be considered to be rigid for simplicity in this example) and lateral forces are distributed to each level in an inverted triangular pattern, In lhis case. all of lhe inelastic deformation is assumed to concenLrate in the lower column. As a resulto the distribution of damage cannot be directly estimated from the ratio of elastic demand to plastic capacity for each element as it might for a single level structure. In the case illustrated in Fig. l. the displacement ductility demand for lhe lower level increases by 50% (i.e.• to six) when the overall systern ductility is four.

Basically, lhe aboye example indicates lhat the ductility demand on the critical elements in a structure will be exacerbated, if the elastic displacement of the structure results appreciably from elements that do not subsequentiy yield. In the aboye exarnple, lhe upper column contributes 40% of the total elastic displacement at the roof level, but it contributes nolhing 10 the incremental inelastic displacements (for the *elasto-peifecr/y* plastic case considered here, at least). The lower column, thus, must displace disproportionately once yielding initiates. This situation worsens if the bent cap (because of its fIexibility and long length) contributes significantly to lateral displacements in the elastic range. If bolh the capand lhe upper column remain elastic foUowing yielding of lhe lower column, ductility demands on the lower column will be even larger than indicated above.

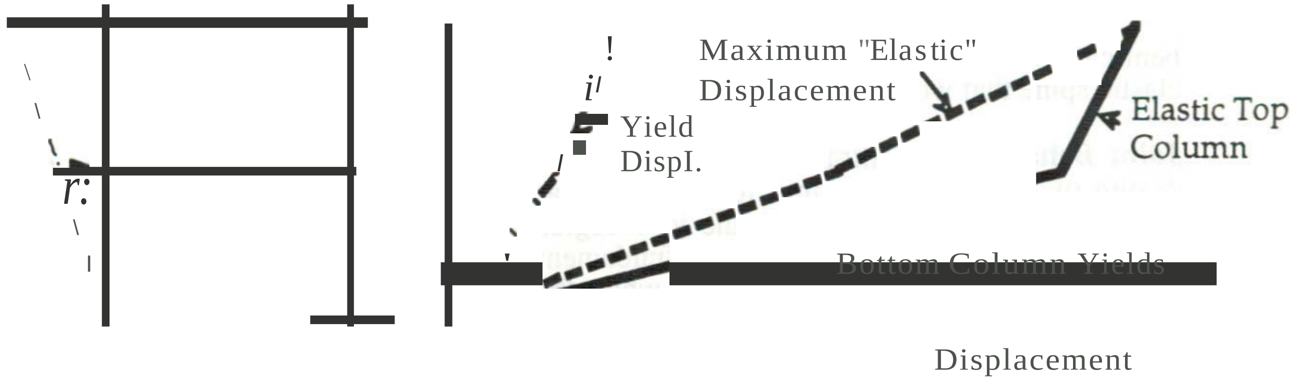


Fig. 1 Effect of Weak Story Mechanism on Apparent Column Ductility Demand

If a situation is considered where lhe stiffness of lhe lower story is increased, for exarnple, by fixing bolh ends of the lower level columns, but the 10p column remains pinned at one end, the distribution of elastic deformations changes significantly (Fig, 2). However, if the lateral shear strength of the structure remains unchanged, inelastic deformations will still occur only in the lower leveI. Reconsidering a situation where the ductility demand on the systems is four, as aboye. lhe displacement ductility demand on lhe lower level column would become 12, or 300% larger lhan expected on the basis of lhe Z values alone.

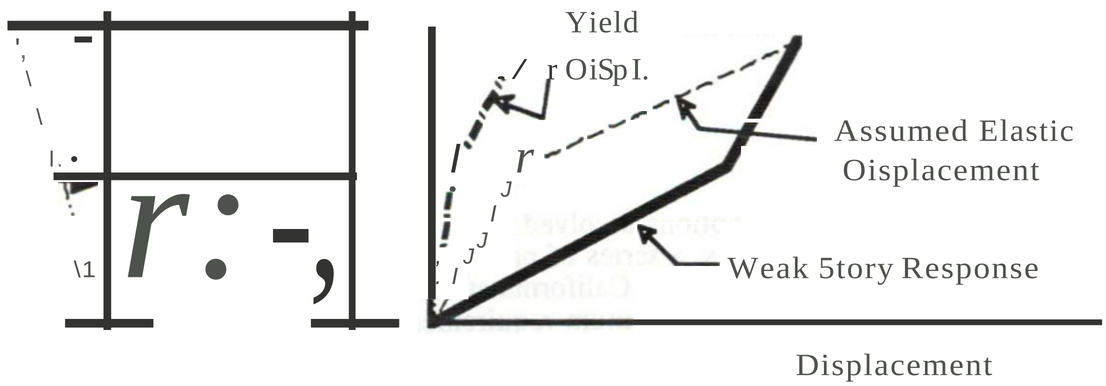


Fig. 2 Weak Story Behavior where the Lower Column that Yields is Four Times Stiffer than Upper Elastic Column

The distribution of damage between two levels may be estirnated on the basis of conventional calculations. or even by hand. However, the precise values obtained depend on many factors including the relative stiffnesses of various elements, the post-yield characterislics of the plastic hinges (e.g., deformalion hardening may permit eventual yielding in locations not expected on the basis of an elastic or simple inelastic analysis), lhe distribution of lateral forces, lhe presumed displacement demand on the systern in lhe inelastic range, the ground motion characteristics and so on. Caution must be exercised in placing too much emphasis on the precision of calculations related lo one particular aspect of this process in víew of the uncertainty of the many assumptions involved and the inherent variability of inelastic response. This suggests a fundamental need ro conservatively assign Z values for design purposes in multi-Ievel viaducts where columns may yield and to carefully detail a11 of the potential plastic hinge regions.

Concentration of damage in one level would not be as likely for cases where the bent caps were designed to yield rather than the columns. The columns would forrn an elastic spine that would tend to equalized displacements in adjacent levels.

Joint Behavior. -- Caltrans' Bridge Design Manual [3] currently does not cover the design of bent cap to column joints other than requiring that the column transverse reinforcement continue through the joint region. Current provisions of the American Concrete Institute are formulated for fundamentally different conditions. That is, ACI provisions are for building structures in which the beams are weaker than the columns. In this case, the ACI stipulates a minimum amount of transverse hoop reinforcement over the depth of the joint and limits the effective horizontal shear stress acting across the joint Since the vertical column longitudinal steel and the horizontal hoop reinforcement remain essentially elastic in such cases, the joint is both strengthened and confined in the horizontal as well as vertical directions.

In the cases of a column that undergoes significant inelastic action, it is doubtful whether the column's vertical reinforcement could provide the joint with sufficient vertical reinforcement and confinement. It is expected that supplemental transverse reinforcement would be required in the vertical (as well as horizontal direction) in the joints, Since the conditions encountered in the double deck viaducts do not conform to those associated with the ACI recommendations, design consultants for the double deck viaducts have typically relied on strut and tie (truss analogy) models to rationalized the amount of vertical and horizontal supplemental reinforcement needed. This provides a direct indication of the load path and assigns reinforcement specifically to various tasks. While the current implementation of the strut and tie model appears conservative, the actual load transfer mechanism and the various inherent assumptions have yet to be fully evaluated.

PROOFTEST

Because of the many assumptions involved in the design of the retrofit concepts utilized for the double deck viaducts, a series of proof tests are being carried out. The first of these was done at the University of California at San Diego. The specimen tested was designed in accordance with the minimum requirements stipulated by Caltrans for the retrofit of the double deck viaducts. As such, structure had a nominal base shear capacity approximately equal to 25% of the dead load weight of the structure, The test specimen incorporated an isolated edge girder. In addition, because of the relatively low lateral loads applied to the structure, the existing transverse bent cap could be strengthened by means of external post-tensioning.

Details of the 1/2 scale UCSD test specimen, loading apparatus and test results can be found in References 6 and 7. The specimen was tested under transverse, longitudinal and diagonal loading excursions somewhat in excess of the design target level of 4. No significant damage was reponed at this level, At larger levels, excessive yielding of the spiral reinforcement in the lower column was noted. A subsequent test was carried out on the same specimen to assess the torsional transfer mechanism across the bent cap between the box girder and the column. No difficulties were noted, and the ultimate capacity of the post-tensioned cap at this location was reportedly satisfactory.

As indicated previously, certain circumstances would suggest the need for an edge girder cast monolithically with the deck. In addition, cases are likely IO occur where post-tensioning alone would not be sufficient to strengthen the transverse bent cap. Because of the differences in details likely in these cases. a second proof test specimen was investigated at the University of California at Berkeley.

The Berkeley proof test specimen was idealized from Bent B8 on the Alemany freeway near the intersection of the 1-280 and I - 101 freeways. The viaduct at this point has a transverse span between column centerlines of approximately 53 tt, and a span between bents in the longitudinal direction of 90 ft, The existing box girder deck has six cells and is 5 ft, -6 in. deep. This bent when retrofit proved to be particularly strong. This was due to a combination of factors including the fundamental periods of the frames near this bent being close to those near the peak in the acceleration spectrum used for design othe relative flexibilíty of adjacent bents resulting in greater load being carried by this bent, and conservatism by the consultant in proportioning and sizing the controlling column. The details of the test model were developed by following the basic design procedures utilized by the designers [8] or by scaling the prototype by appropriate similitude relations.

Because of laboratory height limitations, the test model was based on a 1/3 scale factor. Rather than being curved, the test specimen was assumed to have a straight roadway. The test model was based on the column reinforcement provided in the actual column in the retrofit, The prototype column was 5 ft, - 6 in. in diameter and reinforced with 42 No. 14 Grade 60 bars resulting in a reinforcement ratio of 2.8%. Based on the one-third scaling the model column had 22 No. 4 bars and 20 No. 5 bars. The reinforcement was asymmetrically distributed around the column cross section, in both the prototype and the model (Fig. 3). in two layers to give the structure slightly greater capacity in the transverse direction. Because of uncertainties related to the performance of reinforcement terminated in a joint, as well as practical construction considerations, the column in the upper and lower levels were reinforced the same.

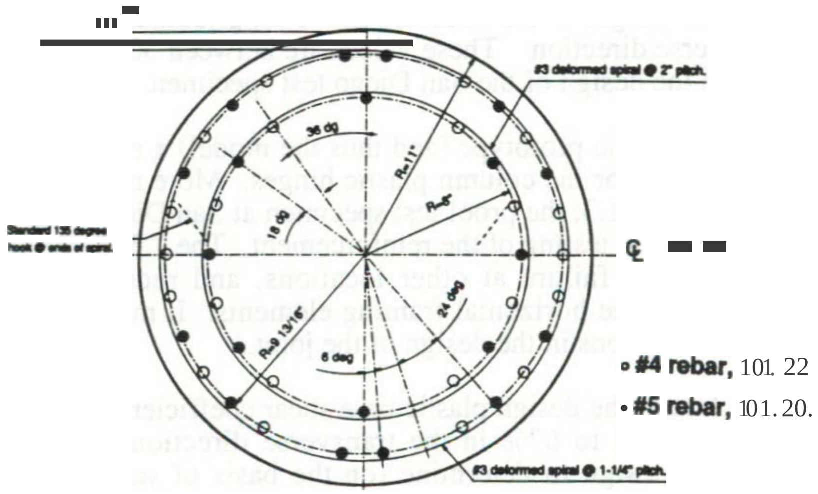


Fig. 3 Column Cross Section Utilized in Proof Test Model

To simplify the design of the modelo the specimen was isolated from the remainder of the structure at the midspan of the bent cap and edge girder as shown in Fig. 4. This idealization represents the point of asymmetry under lateral loading. Since the prototype columns are pinned ended at the top of the upper column and at the bottom of the lower column the model was idealized as having real pins at these locations. Additional information on the model and loading is presented subsequently.

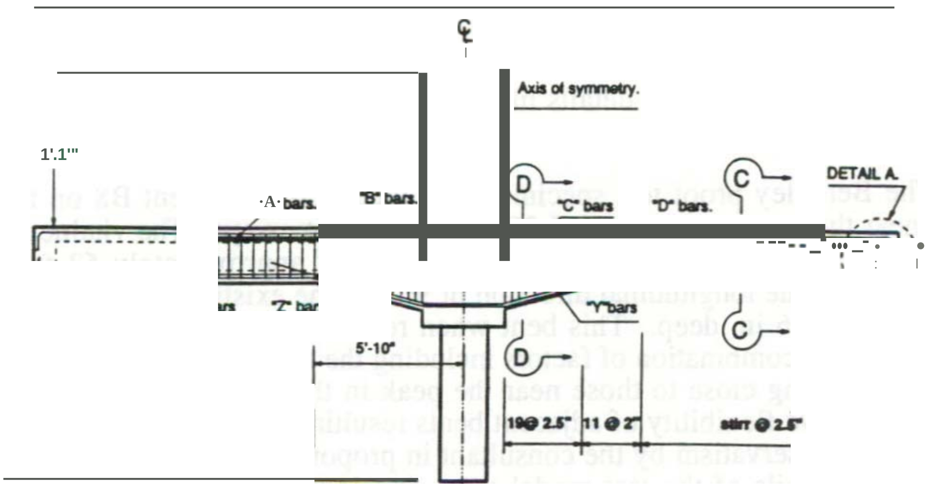


Fig. 4 Longitudinal Side View of Proof Test Specimen

For the design of the model and prototype, a lateral load distribution corresponding to formation of plastic hinges simultaneously in the upper and lower level columns was considered. Since the column in the upper level had the sarne cross section as that utilized in the lower level, this assumed mechanism resulted in a lateral load distribution with nearly a1l (about 95%) of the lateral load applied at the top of the upper column. While this is not realistic from the perspective of dynamic loading on the free standing structure (where about 56% of the lateral load would be applied at lhe top level), it corresponds to the severest load case for the joint, accounts for possible hammering across the expansion joints in the longitudinal direction, and other unanticipated loading conditions. Based on the axial load present in the columns corresponding to the mechanism described aboye. the nominal and plastic shear capacity of the structure was cornputed. The test specimen has a nominal ultimate base shear capacity equal to about 39% of the dead load weight of the structure in the longitudinal direction and 45% of its weight in the transverse direction. These values are between 50 and 80 % larger than the values considered in the design of the San Diego test specimen.

For the design of the prototype (and thus the model) a ratio of plastic to nominal capacity of 1.5 was used for the column plastic hinges. More recent Caltrans guidelines have permitted a factor of 1.3; the proof test specimen at San Diego was based on a factor of 1.1. but required special testing of the reinforcement. The 1.5 factor provides a greater margin of safety against failure at other locations. and reduces the possibility that yielding would occur in the horizontal frarning elements. JI may also justify the use of less conservative assumptions in the design of the joint.

For the 1.5 factor. the design plastic base shear coefficient increased lO 58% in the longitudinal direction and lO 67% in the transverse direction, The arnplified plastic moments are used in design to determine (on lhe basis of statics) the forces that the columns, girders, caps and joints must resist. The increased lateral load capacity may result in very low (or even tensile) axialloads in the lower level columns under transverse loading depending on the amount of dead load assumed present. For the 1.5 factor considered, the moments and shears considered in the design of the Berkeley specimen were nearly 2.5 *times* greater than those used in the San Diego specimen.

Design of New Columns. -- Column shear reinforcement and confinement were computed for the prototype consistent with Caltrans guidelines. However, because of the possible severe distress in the plastic hinge regions, the quantíty of spiral reinforcemenl was increased over Caltrans minimum requirements. The column model was derived from the prototype considering the scale factor. The columns pins were assumed 10 carry no moment as is common practice in designoalthough typical pin connection details are able to develop considerable moment.

Design ot New Longitudinal Edge Girder. -- The longitudinal edge girder was a haunched, open cell box section monolilhically connected to the existing deck. (Figs. 4 and 5). At midspan the edge girder and existing box girder had the same depth, however at the face of-the column, lhe depth of the haunch increased lhe girder depth by about 75%.

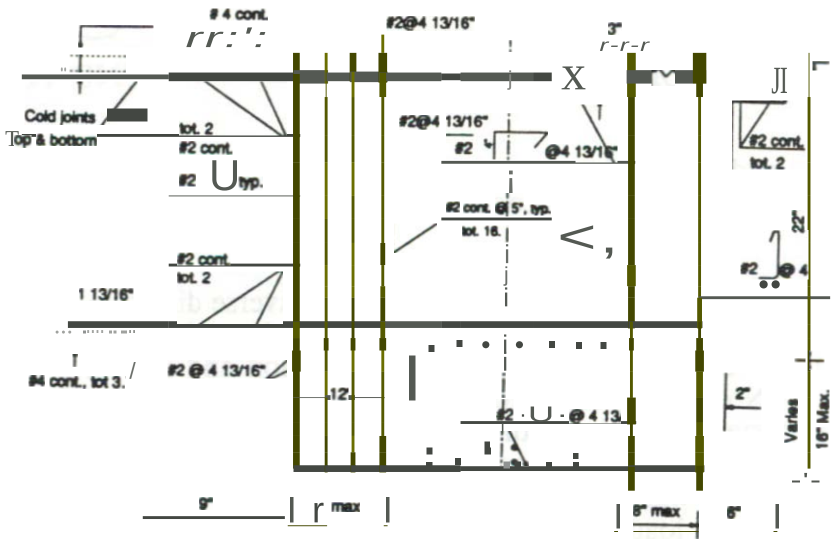


Fig. 5 Cross Section of Hollow Edge Girder Near Face of Column

Moments and shears in the edge girder were determined considering equilibrium at the joint corresponding to the plastic moment of the column sections framing into the joint. No moment was assumed to be transferred to the stems of the existing deck; thus, the cap was not designed to resist torsion, though lhe details used in the cap would be able 10 resist considerable torsion. The flexural reinforcement in lhe haunched portions of the edge girder was computed considering lhe inclination of lhe haunch, and special vertical reinforcement (or diaphragms) were installed at the end of the haunches 10 take the vertical component of the tension or cornpression force developed in the girder at lhose locations. The contribution of an inclined compression strut was considered when evaluating the shear capacity of the edge girder.

Because the edge girder is wider than the column forces in the girder reinforcement not passing through the column core are assumed to be transferred to the column by a three dimensional truss consisting of vertical stirrups that engage the longitudinal reinforcement in the girder, transverse reinforcement running perpendicular to the girder's longitudinal reinforcement and an inclined concrete cornpression strut. This strut action is illustrated in Hg. 6. The tensile forces acting in the reinforcement is assumed to transfer across lhe joint region (due to bond deterioration) and eventually be resisted on the opposite by the inclined compression strut and reinforcement illustrated in Fig. 6.

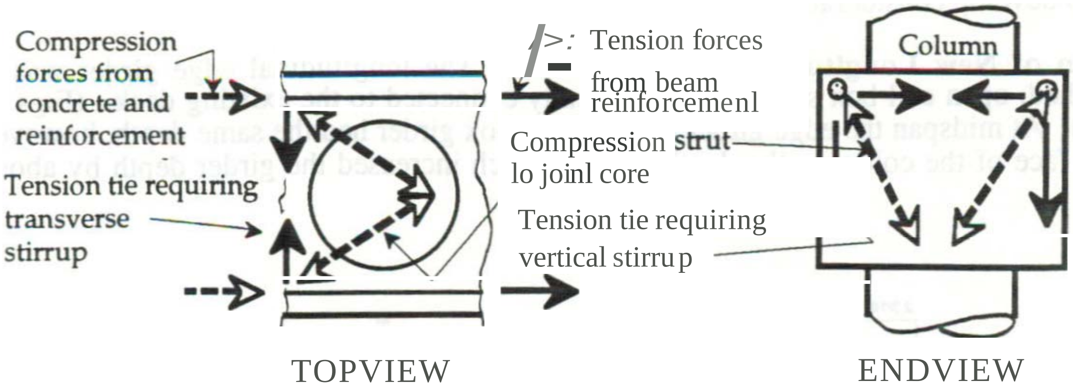


Fig. 6 Inclined Three-Dimensional Strut Mechanism Used to Transfer Force from Girder Reinforcemenl Located Outside of Column Core to Column

Design of Transverse Bent Cap Retrofit -- In the transverse dírection, the existing bent cap was heavily reinforced, but it was difficult to continue thís steel into the heavily reinforced column. Because of a change in the location and size of the column, the existing Grade 40 No. 18 cap bars had to be CUl and extended using smaller diameter Grade 60 reinforcement. Similarly, the positive moment reinforcement in the cap was inadequately anchored in the joint and would also have to be extended. Because the reliability of the splices was uncenain and the congestion of the column reinforcement made it difficult to resist load with the existing concrete section, the designer decided to terminate a11 of the reinforcement in the existing bent cap away from the new column.

The moment transfer from the bent cap to the column was achieved by adding reinforced concrete bolsters to the sides of the existing bent cap (Fig. 7). These were reinforced with mild reinforcement (No. II bars) as well as post-tensioning steel. However, about 70% of the moment capacity was provided by the post-rensioníng. The reinforcement and post-tensioning extended in the bolsters past lhe side of the column coreo A three dimensional strut and tie model (similar lO Fig. 6) was used to rationalize the force transfer from the cap through the joint and into the column.

The mild reinforcement in the cap terminated in the joint without a hook at its end (Fig. 8). Because the anchorage might deteriorate under cyclic loading, hooked bar or flat plate anchorages would be preferable. Similarly, the sudden change in reinforcement, where the longitudinal reinforcement in the existing bent cap is terminated, would suggest that a nominal amount of reinforcement (maybe 25%) should continue from the existing cap into the new joint coreo Because of the conservatism inherent in the determination of the design forces for the cap. it was decided not to incorporate these modifications in the proof test specimen.

The bend cap post-tensíoning was curved inward around a vertical axis toward the longitudinal axis of the bent cap to help confine the joint. However, post-tensioning configured in this manner develops significant splitting (bursting) stresses at the outer edge of the joint. As a result, considerable reinforcement was required across the outer edge of the joint parallel to: the longitudinal axis (see Fig. 8). This was added to reinforcement required for other purposes.

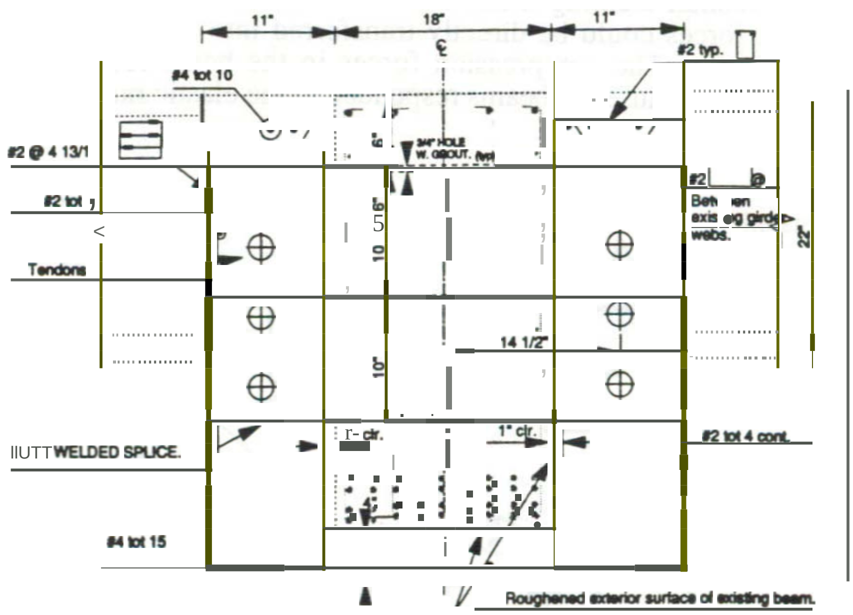


Fig. 7 Cross Section of Bent Cap Showing New Concrete Bolsters (Note: Reinforcement in Existing Bent Cap is Terminated at Interior Face of New Edge Girder)

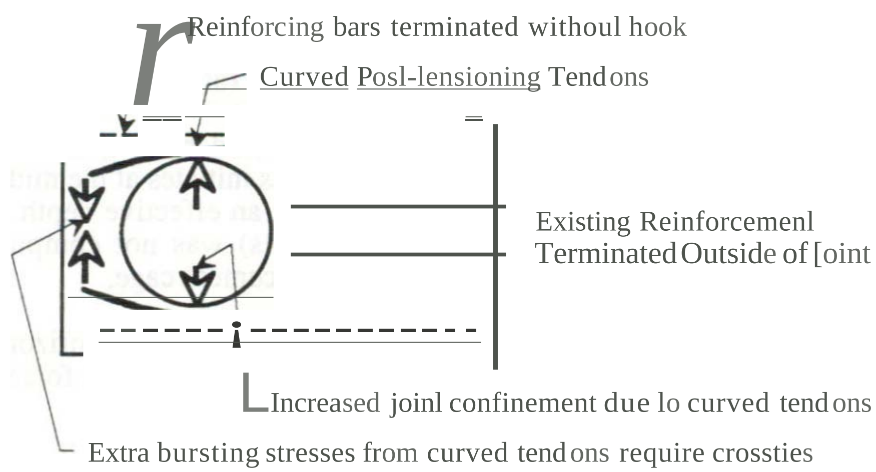
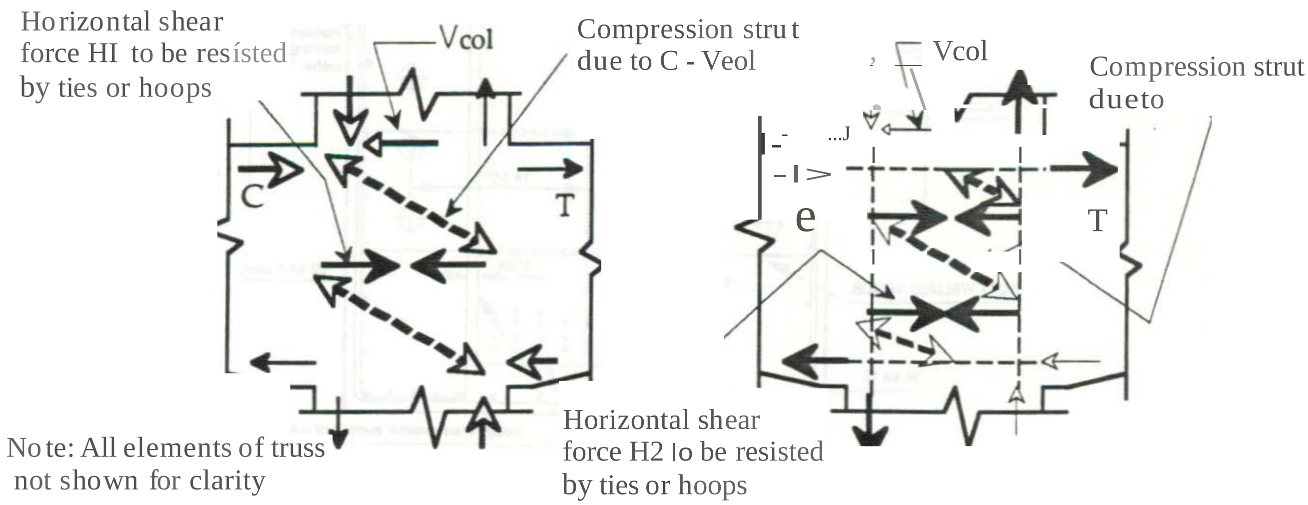


Fig. 8 Schematic Top View of Bent Cap 10 Joint Connection Showing Termination of Reinforcement and Post-tensioning, and Bursting Reinforcement

Joint Design. -- The joint was designed for the plastic moments and shears from the columns framing into it, Moments and shears in the girders or bent cap were determined considering equilibrium at the joint (and dead loads). Because axial compression in the columns improves the shear strength of the joint, the analyses were carried out for a reduced (by 20%) dead load. This reduction was intended to account for vertical accelerations of the ground and other factors.

The strut and tie method was used by the designer [8. 9] to determine the reinforcement of in the joint. Since the problem is statically indeterminate. simplified and generally conservative (additive) assumptions were employed in the analysis of the resulting three dimensional trusses assumed to transfer forces from the column through the joint into the horizontal framing members. The method employed assumed that concrete compression forces could be directly transferred into the joint as an inclined compression strut (Fig 9). The compression forces in the beam reinforcement were assumed IO be negligible (because the beams responded in the elastic range).



a. Transfer of Compression Force from Positive Moment Girder

b. Transfer of Tensile Forces from Negative Moment Girder

Fig. 9 Schematic Illustration of Truss Models used to Transfer Forces from the top of the Edge Girder into the Column (after Ref. 8)

Tensile forces in the beam reinforcement are resisted by a truss mechanism consisting of steel ties and inclined concrete struts, The truss initiates at the mid-depth of the column (Fig. 9). Because the column section is round, an effective depth (between centroids of the compression and tension stress resultants) was not computed, but estirnated to be 80% of the outer diarneter of column reinforcement cage.

Figure 9 illustrates the trusses considered to determine the total horizontal shear force required to be carried by reinforcement in the joint region. The total force required is given by

$$H = HI + H2$$

where HI, corresponding to the girder introducing a compression force on the top face of the joint, is given by:

$$HI = (Mljd_g - Vcol)(d_g - d_{col}\tan\alpha)/(d_{col}\tan\alpha)$$

except where Vcol> Mljd, in which case HI is talcen to be zero: and H2, corresponding to the girder causing tensile forces at the top face of the joint, is given by:

$$H2 = Mljd_g + C - Vcol)(dg)/(dcoltanll)$$

except where Vcol> Mljd$_g$ (compression side), in which case H2 is taken to be equal to Mljdg(dg)/(dcoltana), and

in which M is the moment acting in the girder in question;
jd$_g$ is the effective depth of the girder between the centroids of the compression and tension stress resultams:
d$_g$ is the depth of the girder from the centroid of the tensile steel to the compression face;
dcol is the effective depth of the column, taken herein as 80% of the diameter of the outer ring of column reinforcement;
α is the inclination of the compression strut (taken as 55°); and
other tenns are as defined in the figure.

This total horizontal force is assumed to be partially resisted by horizontal welded hoops (no! spirals) surrounding the column core in the joint region. The amount resisted by the outer set of hoops is given by the product of the number of hoops provided over the height of the joint, the area of each hoop bar, the yield stress of the steel (60 ksi) and the number of legs on each hoop (2), multiplied by an efficiency factor of $\pi/4$ lO account for the varying inclinatíon of the hoops as they cross an inclined crack running through the joint. The inner hoops are not considered by the designer to be as effective [8], since they encircle fewer longitudinal column bars. The vertical component of the force resisted by the hoops is assumed to be transferred lO the column bars in the strut and tie modeI. Fewer column bars results in less force that can be transferred to the hoops. Thus, the efficiency factor, γ, for the interior hoops is given by:

$$\gamma = (\pi/4)\,(d_{col_{inner}}/d_{col_{outer}})\,(S_{inner}/s_{outer})\,(N_{inner}/N_{outer})r$$

in which the subscripts inner and outer refer to the inner and outer rings of r reinforcement or hoops,
s is the spacing of the hoops; and
N is the number of longitudinal bars in a ringo

The remaining horizontal force is transferred to the cross ties extending across the entire joint in the direction of the applied load. Each end of the cross tie is terminated with a 90° hook. The contribution of the cross ties to the horizontal force required to be resisted in the joint is given by the product of the area of the cross tie, the yield strength (60 ksi) and the number of cross ties provided. Because of difficulties in inserting these through the column core, most of the cross ties are distributed vertically over the height of the stern of the edge girder.

The vertical cornponent of the tensile inclined strut is given by the larger value obtained from the following lWO equations:

$$V = (Mljdg(\text{compression side}) + Mljdg(\text{lension side}) - Vcol)\ \tan u - 0.8\ CCtop$$

and

$$V = (Mljdg(\text{compression side}) + Mljdg(\text{tension side}) - Vcol)\ \tan o; - 0.8\ [CcbOl - Vg]$$

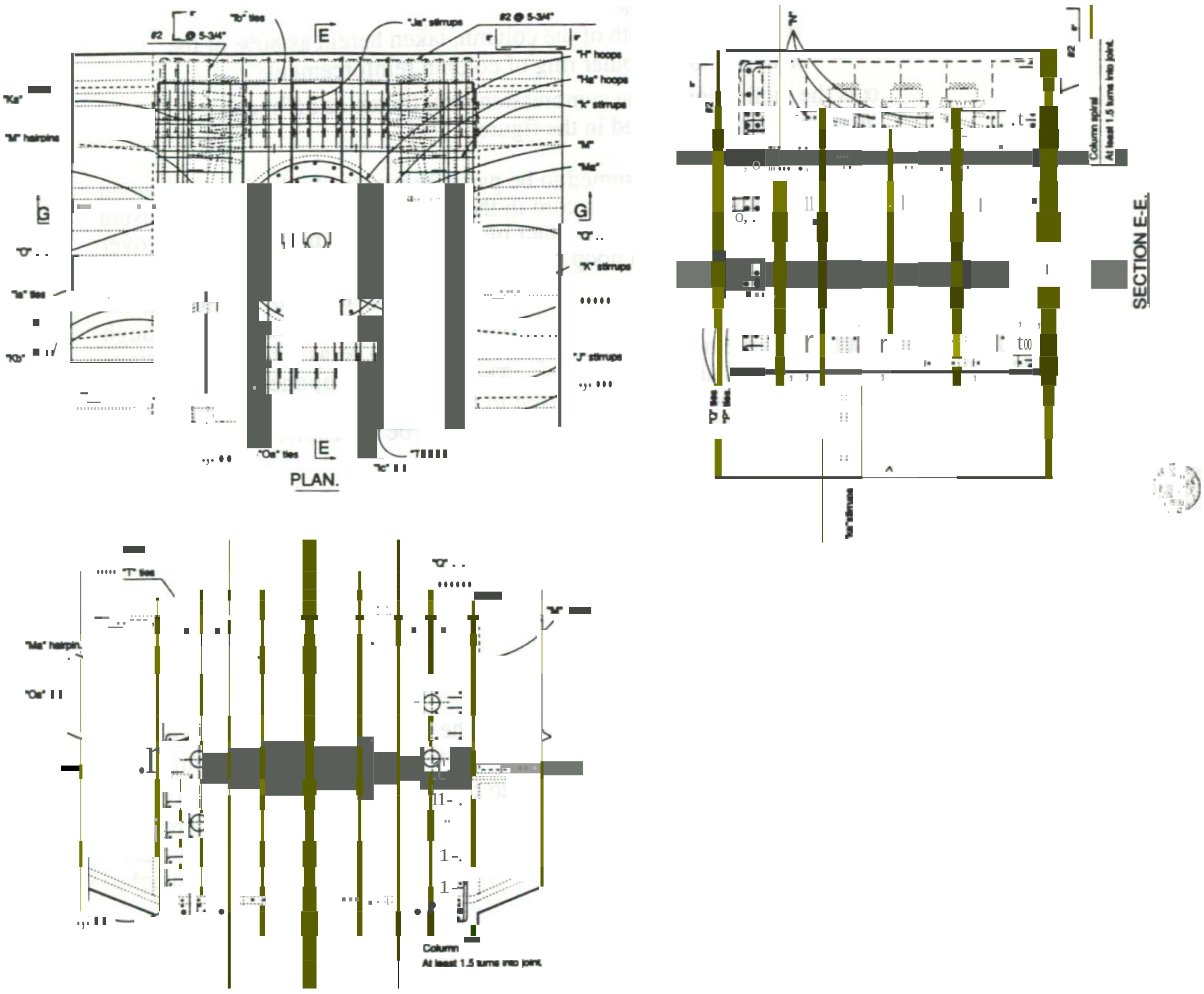


Fig. 10 Typical Joint Details for Joint Region

in which C_{ClOp} corresponds 10 lhe compression force resultant in the IOp column resulting from load reversal wilh the steel having an effective strength of 1.55 times the yield strength,

CCbot corresponds to the compression force resultant in lhe bottom column resulting from load reversal wilh the steel having an effective strength of 1.55 times the yield strength, and

- Vg is lhe sum of alJ venical dead load shears tributary lO the caps and girders framing into lhe joint.

An area of steel is provided in the joint such that the aboye force is developed by lhe total area of vertical reinforcemenl provided in lhe joint, lhe yield strength of lhe bars (60 ksi) and an efficiency factor. For the vertical bars provided inside the column core an efficiency of 100% was assumed. For bars outside of the column coreo an efficiency of 50% was assumed. AH bars were terrninated wilh a hook on each end. In the case of lhe bars added within the column coreo they extended a shon distance into the column and terrninated with a 180° hook that engages a short transverse bar of nominal size.

As a result of the high design forces, conservative assumptions regarding the ratio of plastic 10 nominal flexural capacities, and efficiency factors less than unity for the vertical and horizontal shear reínforcement, the joint had a large amount of transverse shear reinforcemenl. This lead lO significant constructability problems that could have been panially eliminated by using a larger column and joint cross section, and different anchorage details for the cross ties.

The design of the joint was also carried out in lhe transverse direction. This was similar to the rnethod described aboye except the significant contribution of the pOS1-tensioning to the shear resistance of the joint was included. For the sake of brevity this developmenl will not be presented here. The interested reader is referred lo Reference 8.

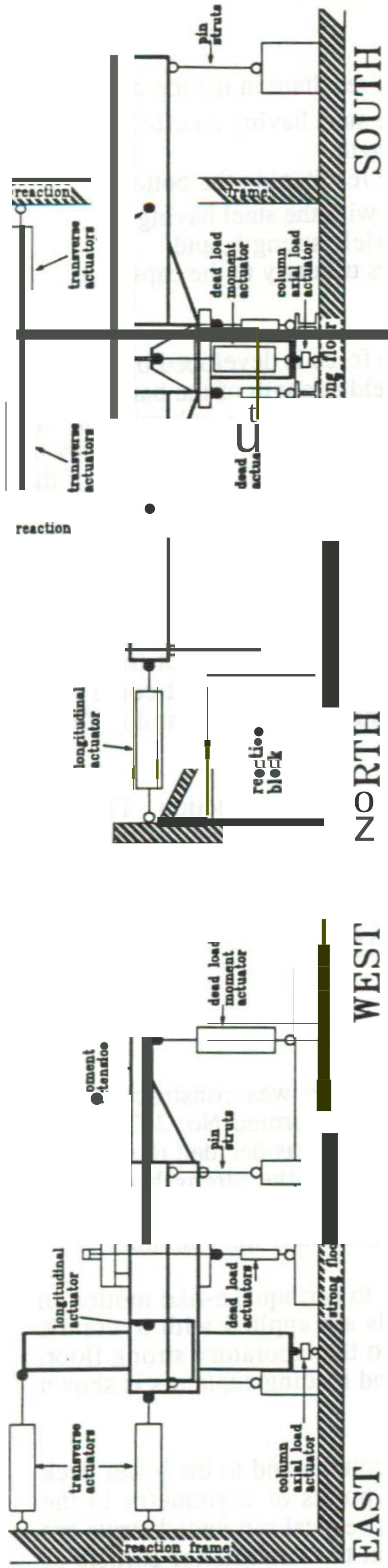
Typical sections through the joint are shown in Fig. JO.

SPECIMEN CONSTRUCTION AND TESTING

In as much as possible al one-third scale, the specimen was constructed as it would be in the field. Because of scaling limitations, sorne deforrned No. 2 Grade 60 bars were used in the as par! of the existing deck. In general. il was decided 10 scale the deck lO match the stiffness of the flexural reinforcement and the strength of shear reinforcemenl. Construction sequence and pour joints were generally as they would be in the actual structure. Construction of the specimen 100k approxirnately three rnonths.

Loading Apparatus. -- The response of the test specimen lO earthquake-Iike motion in each principal plan direction is being assessed. The loads are applied with hydraulic actuators attached to rigid reactíon frames post-tensioned to the laboratory strong floor. The general arrangement of the specirnen, reaction frame and loading actuators is shown in Fig. 11.

The lateralloads are applied to the top of the upper column and 10 the lower deck level, Because the edges of the specimen correspond to points of asymmetry in the elastic deflected shape of lhe specimen under lateral loads, venical pin ended struts are provided at the ends of the girders and bent cap as well as at the free interior corners of



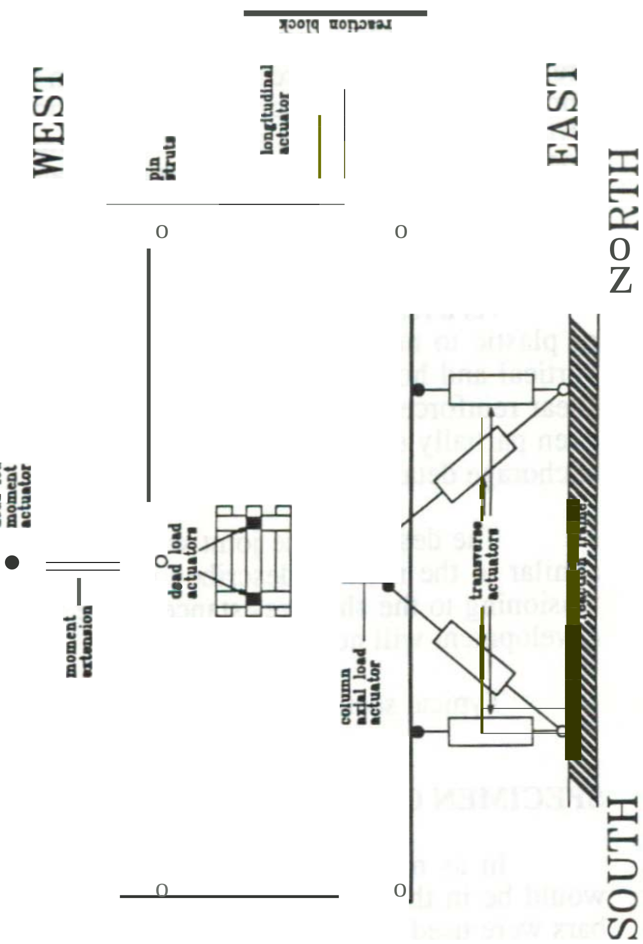


Fig. 11 Test Setup

the deck. Because of lhe finite length of lhe struts, a specially designed vertical actuator is provided under the column 10 maintain the axial load in the column al a proper leve!.

Gravity loads are sirnulated using a combination of concrete and lead weights as weIl as vertically oriented hydraulic actuators. The column is supponed on an actuator that cornpensates for strul arching, inelastic elongation of the plastic hinge region in the lower colurnn, and overtuming moment effects due lo longitudinal and transverse loading. A post-tensioning rod extends up through the center of the column and is loaded with a hydraulic actuator al íts IOp in order 10 maintain the axial load in the columns al the proper leve!.

Because of the need lO sirnulate realistically lhe distribution of moment and shear in the bent cap, two vertical actuators are used lO load the bent cap al lhe interior stems of the bridge deck. In addition, lO replicate the positíve moment al lhe midspan of the bent cap due to dead load. a steel outrigger is cantilevered from lhe edge of the specimen and a constant vertical upward force is applied during the test,

Al the lower deck level lWO actuators are used lO impose specified displacements in the transverse direction and 10 limit torsion, A single displacernent controIled actuator is used in the longitudinal direction al the deck leve!.

Two actuators are connected in a horizontal plane lO the top of the column. The forces in these actuators are control1ed rather lhan displacernent as done al the lower leve!. If displacements were controIled al both levels, changes in deflected shape (that rnight occur as a result of concentrated yielding in one of lhe columns) could not be detected. In lhis case. the force in the upper level actuators is based on lhe elastic mode shape computed for the specimen, the mass distribution al the lWO levels, and lhe column heights. For the test specimen, lhe force at the 10p level should be about 1.25 times the force applied al the lower level in the same direction. This force is determined on-line during lhe test.

Because the actuators al the upper level are rotated with respect 10 the principal axes of lhe structure, a geometric correction must also be applied. Since the top of the column displaces during the test, the geornetric correction depends on the displaced configuration of the specimen al each step in the test. Furthermore, the forces measured in the load ceIls of the actuators auached 10 the lower deck will not be oriented in the principal direction of lhe deck once the structure is subjected lO large bi-directional motions. Thus, lhese forces rnust also be corrected prior lO being used lO determine lhe lateral forces 10 be imposed al the 10p of the columns. A dedicated high speed microprocessor is used lo make these coordinate transformations and control the specimen during the test, The microprocessor requires continual input of the measured deck level actuator loads, lhe measurcd column IOp and deck level displacements. The microprocessor provides the actuator control apparatus wilh analog output signals for controlling the force al the top of the column.

A total of 10 hydraulic actuators are employed in the test, These are controIled by a microcomputer based control system.

Instrumentation and Data Acquisition. -- A wide variety of instruments and transducers are instal1ed on lhe specimen 10 record response during the test, These instruments inelude strain gages, load ceIls and displacemenl transducers. Displacemenl transducers are installed lO measure global displacements, joint deforrnations and rotations, displaced shapes of lhe girders and bent cap (ineluding twist), and the flexural and shear defonnations in the columns. More than 250 channels of dala are acquired during a test.

Loading Sequence.•• A wide variety of loading histories were considered for the test. These included simple diagonal load paths, cloverleaf paneros and other sequences that would help identify the mechanical characteristics of the structure. The loading history selected is shown in Fig. 12.

Each step of loading is divided into three phases. In the fist phase the structure is subjected to a simple displacernent cycle in the transverse direclion (longitudinal displacement at the lower deck is restrained during this excursion) followed by a simple cyclic excursion in the longitudinal direction. The second phase involves two cycles of bi-directionalloading in a square panero; one clockwise and one counter clockwise. The maximum amplitude of the bi-directional excursión is the same as that for the uni-directional excursions; i.e., the projected displacement in the transverse direction is reduced to about 70% of that imposed during the unidireclional excursions. The third phase repeats the fírst phase so that deterioration of mechanical properties may be easily detected,

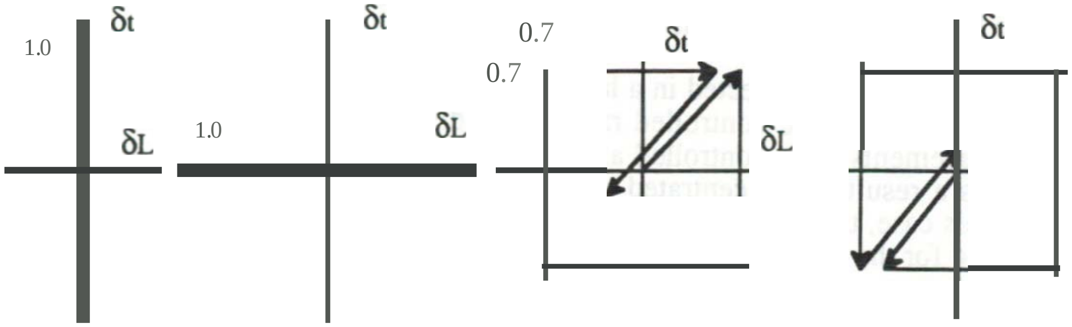


Fig. 12 Displacement History for Proof Test Specirnen

The displacernent history applies only to the lower leve!. The forces applied to the top of the column will correspond to those associated with lhe first mode shape and the forces required at the lower level to develop lhe specified displacements. Because of the complexity of bi-directional response, the upper column moves bi-directionally even under uni-directional displacements at the lower leve!.

The yield displacement is estimated to be approximately 0.8 inches at the lower deck leve!. As such, the specimen will be displaced to 0.1, 0.25, 0.5. 0.75 and I inch lateral displacement amplitudes, and 10 nominal displacement ductilities of 2, 3, 4. and so on. It is expected that the test will be interrupted prior 10 the onset of major damage for repair. This will pennit the effects of minor earthquake repairs to restore the structure's mechanical properties. The test will then be continued until failure. If the specimen appears lo be failing by disintegration of the column plastic hinge and the joint is relatively intact, the column will be strengthened by steel jacketing and testing will be resumed to investigare the ultimare behavior of the joint.

Testing has begun and very low level tests sequences have been completed. Testing should be completed during June 1992.

ANALYSIS RESULTS

To assist in performing the tests and provide insight in to the behavior of the structure, several preliminary analyses have been carried out. These relate lo prediction of deforrnation capacity of the structure in a static sense and simulation of the inelastic response of the structure under statíc and dynamic excitations.

Deformation Capacity. -- An approximate estimate of lateral displacements were made of the displacement of the systern at yield and at failure of the lower column plastic hinge. Element stiffnesses in the elastic range were based on approximate cracked stiffness values (approximately $I_{g/2}$). Based on the elastic distributions of moments and the capacities of various sections, inelastic action should be limited to the top of the lower level column. For the ultimate condition, the lenglh of the plastic hinge in the column was estimated as half the diameter of the column, and the maximum plastic rotation of the hinge was estimated (see Ref. 10) to be about 0.038 radians. Using simple portal analysis procedures, the displacement al first yield in the longitudinal direction is computed to be nearly 1.2 inches at the top of the upper level and 65% of this value at the lower deck. At the ultimare condition, the displacement at lhe top increases to 3.6 inches with the displacement at the lower level equal to about 86% of this value. Similar values were obtained in the transverse direction.

It is clear that the displacements concentrate in the lower level, Dividing the displacement at the top at uitimate by the corresponding value at first yield results in an approximate overall systern ductility capacity of 3.0. If the column itself were analyzed as a free standing cantilever, a displacement ductility capacity of about 5.5 would be predicted. As indicated in the beginning of the paper, the concentration of damage in one element reduces the overall ductility capacity of the structure (altematively, it increases the ductility demand on the damaged element for a given system ductility).

Response Analysis. -- To assess the potential dynamic response of the structure, a series of static and dynamic analyses were performed on an analytical model of the specimen using the ANSR computer program [11]. The analytical model consisted only of the columns, edge girders and bent cap. Torsional response about a vertical axis was restrained. The foundation was assumed to be rigid, and the dead load reactive masses tributary to each level were included in the analysis. Viscous damping equal to about 5 % of critical in the model's first and third modes was assumed. The columns were idealized using a generalized plastic hinge model that has a parabolic shaped axial load - bending moment interactíon curve that matches that for the actual column. However, the hysteretic model employed for lhese preliminary analyses is elasto-plastic and no fixed end rotations are considered. The joint is assumed to be rigid in the analyses.

Because the model is one-lhird the size of the prototype, the periods of the structure are one-third of the values that would be obtained at full scale (Le.• 0.22 sec in the transverse direction and 0.25 seco in the longitudinal direction). To maintain proper relations between the frequency characteristics of the specimen and the earthquake record, the time scale for the record was also reduced by a factor of 1/3. Thus, the earthquake record will only be one-lhird as long as it should be. The displacements obtained in the analyses will be one-third of those expected in a full size structure.

Static Loading. --Under monotonically increasing lateral loads the displaced shape changes from a situation where lhe upper story contributes a significant amount of the total displacement to the weak story case where the displacements in the lower level dominate (Fig. l. Because of the 3% defonnation hardening, the defonnations increase in the upper leveis after fonnation of a plastic hinge in the lower level.

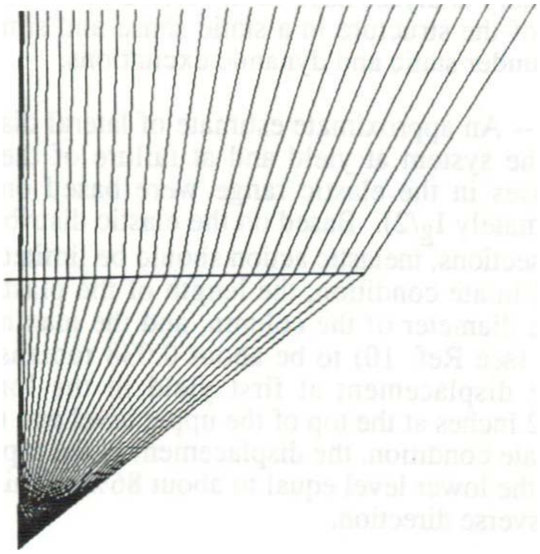


Fig. 13 Displaced Shapes Obtained in
Static Inelastic Analysis (Transverse Direction)

Unidirectional Dynamic Loadlng, -- The analytical model of the test specimen was subjected to the Taft earthquake record (this is a firm soil record like that at the design site), Results for the Taft record, applied in the transverse direction and scaled to have a spectral acceleration four times larger than the yield capacity of the model (i.e., roughly speaking a Z factor of 4), are shown in Fig. 14. Peak displacements at the top are 2.53 inches corresponding to a global displacement ductility of about 2.2. Displacements at the lower level are nearly 90% of this value, clearly demonstrating the fonnation of a weak story mechanism. The base shear time history in Fig. 14 indicates the effectiveness of the yielding of the lower column in limiting the forces that can be transferred lO the structure. The peaks of the pulses in the shear history are limited to approximately one-fourth of tbe value that would be achieved had the structure remained elastic. As noted aboye, the duration of the record has been compressed to one-third of its normal value.

The hysteretic loops shown in Fig. 14 indicate that a large number of nonlinear excursions were developed by the systern. The hysteresis plot on the right side is for the lower level and represents the response of the yielding column (plus the effects of joint rotation). The plot on the left shows the top level displacement as a function of base shear. This represents the overall response of the structure. The erratic (figure eight shaped) nature of sorne of the cycles is a result of higher mode effects, not present in the static analyses or in the test

The intensity of the ground motion was increased. This might be reasonable in view of the fact tbat the structure was nearly 2.5 times stronger than the specimen tested at UeSD. In addition, current design guidelines for new rnulti-column viaducts allow Z values between 4 and 8 depending on the structural periodo For the prototype periods between 0.66 and 0.75 seconds (i.e., three times 0.22 and 0.25 seconds), Z values of nearly eight could be used in design of new non-lifeline multi-colurnn structures.

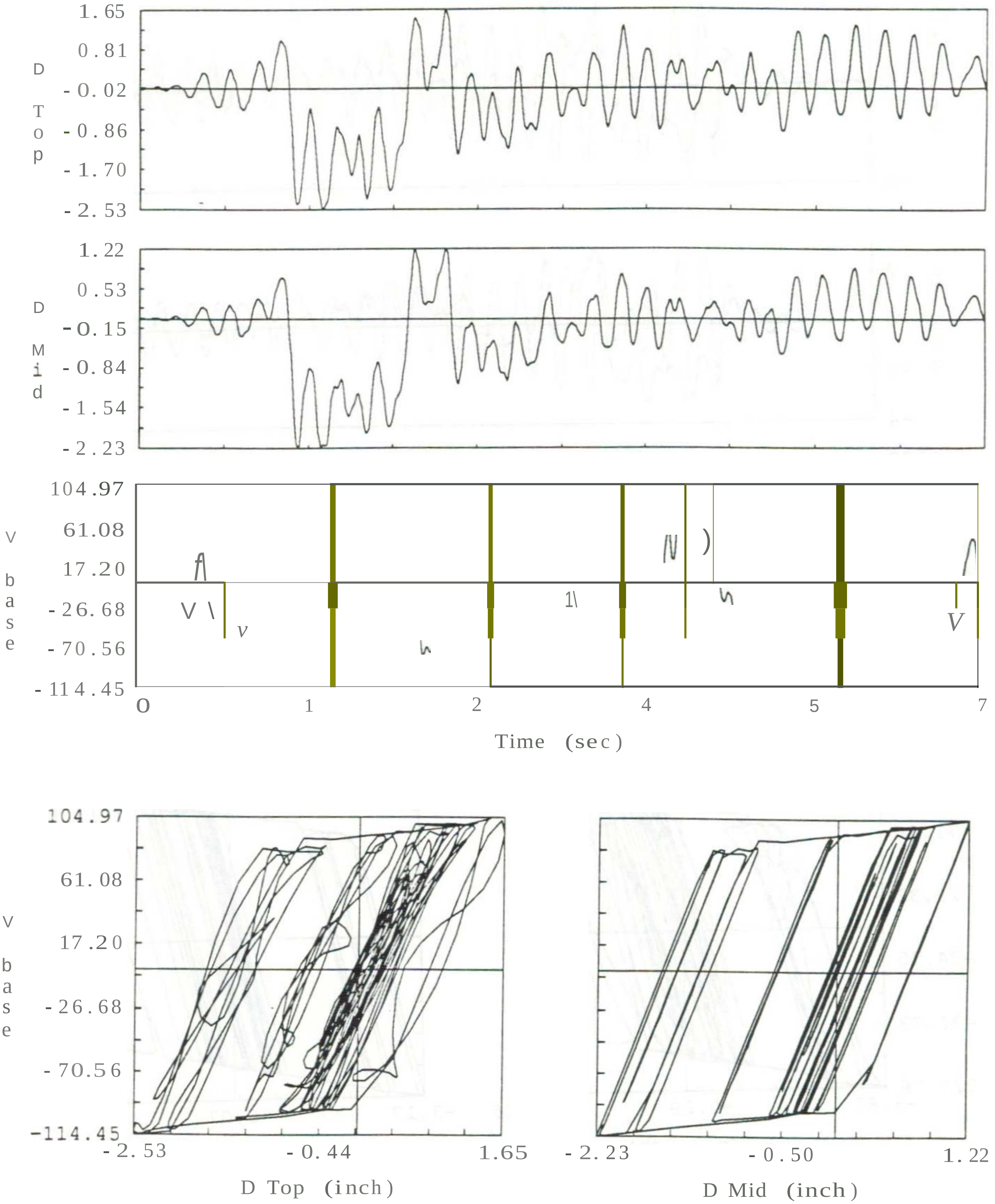
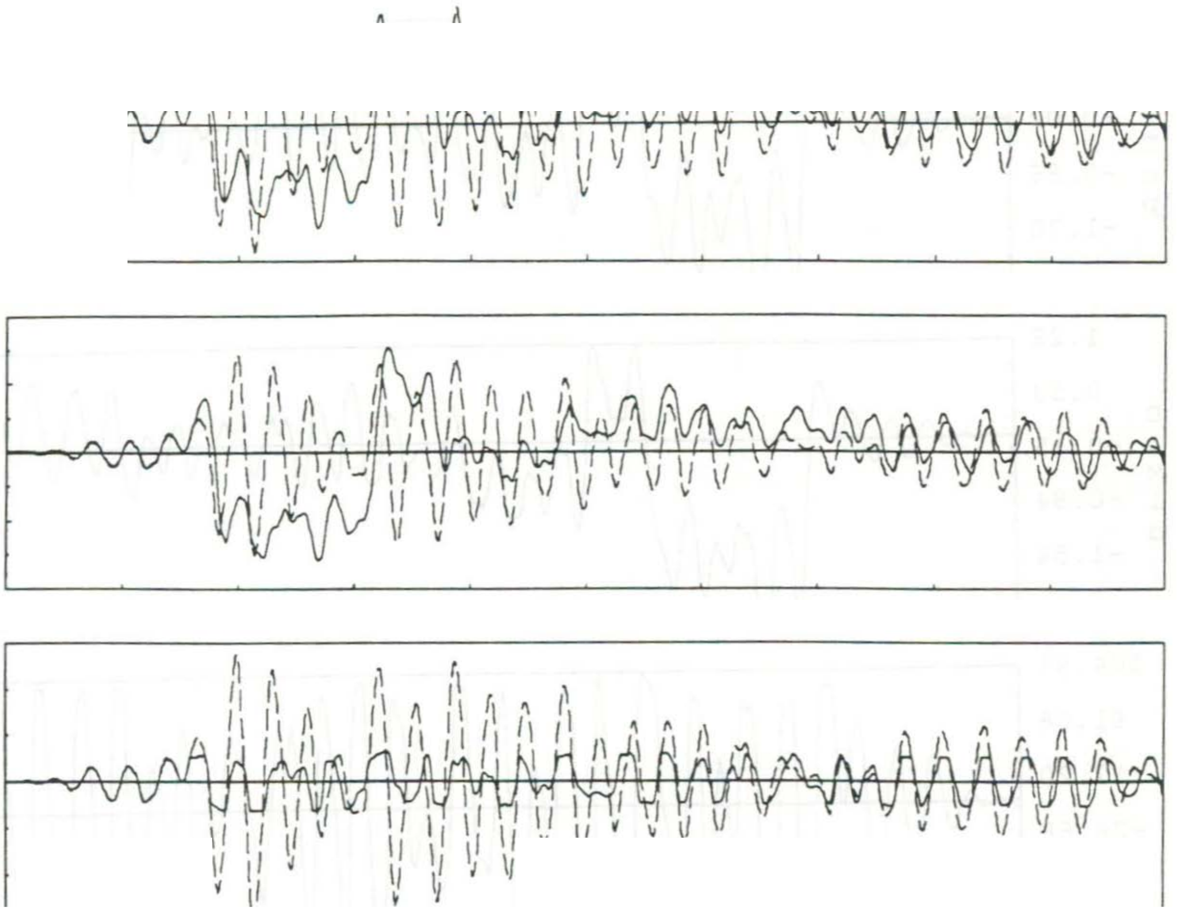


Fig. 14 Inelastic Response of lhe Analytical Mode1 to the Taft Earthquake Record in lhe Transverse Direction. Earthquake Record scaled to correspond to Z = 4.



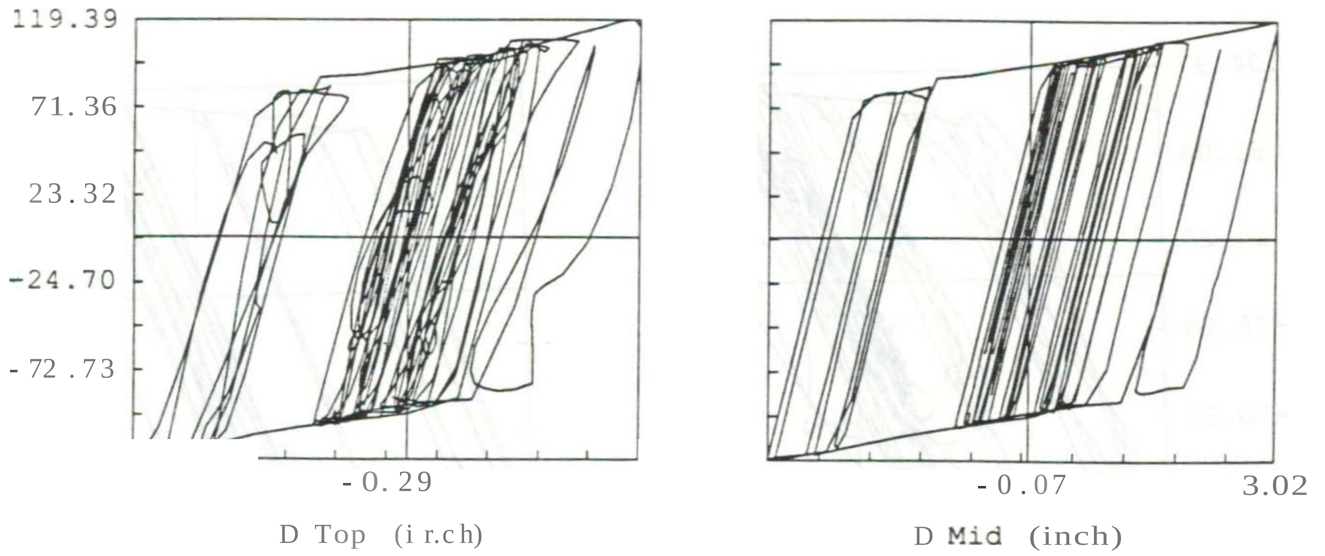


Fig. 15 Analytical Response of the Model to the Taft Earthquake Record in the Transverse Direction. Earthquake Record Scaled to Correspond to Z =6. lnelastic Response Shown as Solid Unes; Elastic Response Shown as Dashed Lines.

Figure 15 shows the response for the Taft earthquake scaled to a Z value of 6. The maximum displacements increase by 52% to 3.85 inches, slightly in excess of the previously estimated ultimate displacement capacity. It should be noted that the inelastic analysis model does not model failure or degradation of the plastic hinge. Also ploned in this figure are the elastic response results. While the force levels are substantially reduced by the yielding, and the panero of inelastic displacements differs from that of the elastic response. the maximum displacements are similar.

The similarity of elastic and inelastic peak displacements has been noted by many researchers for single and multiple degree of freedom systems so long as the period of the structure is longer than the predominant period of the ground. This can be seen in Fig. 16. where the maximum elastic and inelastic displacements at the top of the structure are plotted for the Taft record scaled to various Z values. It can be seen that especially at the top of the structure, these values are nearly identical. At the lower level, the values begin lO separate for Z values exceeding 8.

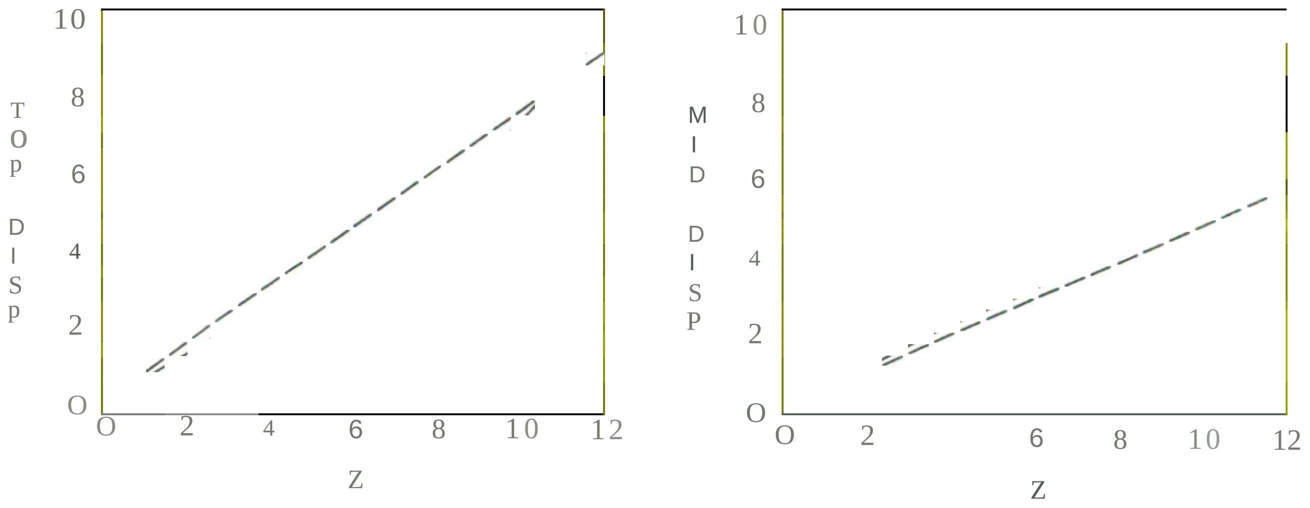


Fig. 16 Top and Mid-level Lateral Displacement for Different Intensities of Shaking (Z values), Solid Lines Correspond to Inelastic Response; Dashed Lines Correspond to Elastic Response.

The reasons for this behavior can be seen in Fig. 17 which shows the envelop of máximum elastic and inelastic displacements for various Z values. The shape of the elastic envelop remains the sarne, with the ratio of displacements in the upper level to the lower level a constant. In the inelastic response. the deformed shape shows a gradual transformation to a weak story collapse mechanism. For large Z values, nearly all of the lateral displacement occurs in the lower leve!. Thus, the distribution of damage is not easy to predict considering the results of the elastic analysis. But the top displacement is reasonably predicted by the elastic analysis.

The ability of a Z factor to control the inelastic response of a structure for periods longer than the predominant period of the ground motion is illustrated in Fig. 18. This figure shows the maximum displacement ductility developed for single degree of freedom structures having 5% viscous damping and stiffness degrading mechanical propenies. In Part a of the figure. values obtained for different Z values are shown as a function of period for the nonh - south component of the 1940 El Centro earthquake record. It is seen that the ductility value developed is similar (though by no means identical) to the Z value for periods greater than 0.5 seconds. For periods less than 0.5 seconds, ductility demands rapidly increase. In the region on amplified acceleration the maximum displacements can be estimated from elastic values considering conservation of energy. This results in lhe following expression:

$$\delta_{inelastic} = \mu(2\mu-1)^{-0.5}\,\delta_{elastic}$$

in which u is lhe displacement ductility of lhe system. In the short period range it is not possible substitute Z for u in this equation. Rather, it is possible to show for an elasto-perfectly plastic system lhat Z = $(2\mu-1)^{-0.5}$. In this case. lhe inelastic displacement may be estimated as follows:

$$\mu = (Z^2-1)/2$$

and

$$\delta_{inelastic} = (Z2\text{-}1)/2Z\ \delta_{elastic} \approx 0.5Z\delta_{elastic}$$

where the approximation is good for large values of Z.

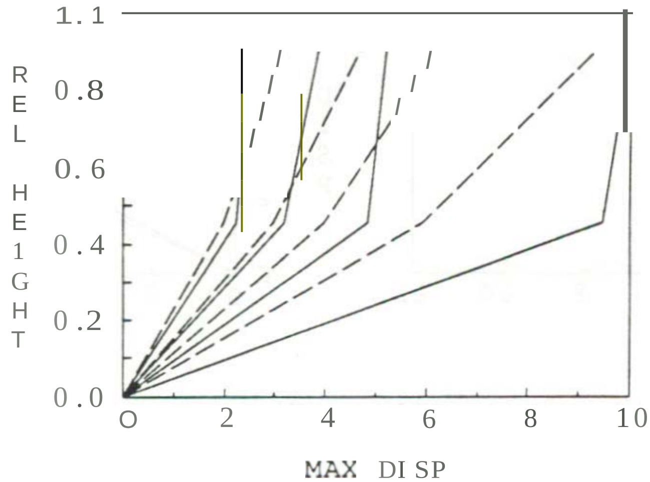
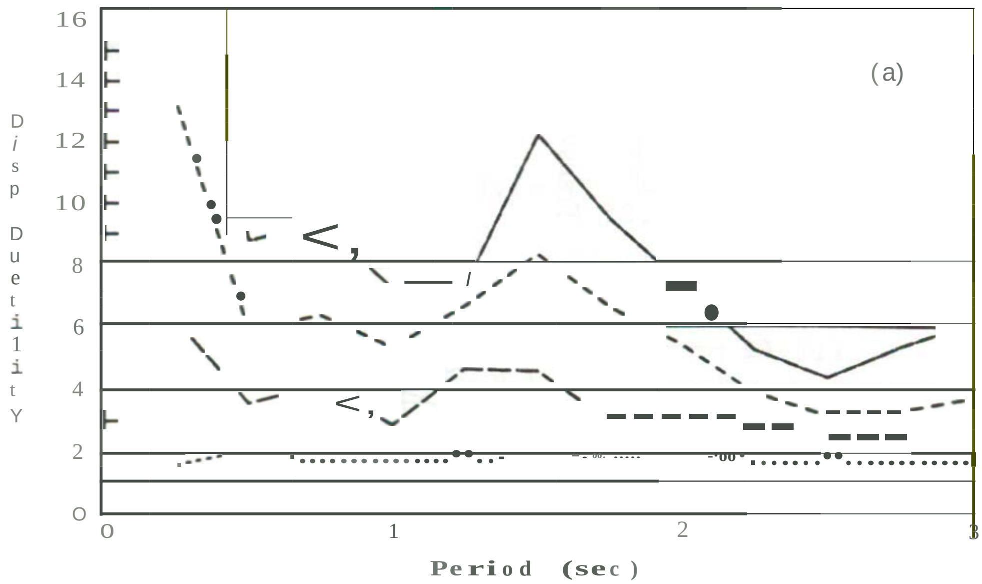


Fig. 17 Envelop of Lateral Displacements of Analytical Model for Z values rangin" from 4 (on Left) to 12 on Right) for Taft earthquake. Sohd Lines Correspond 10 Inelastic Response; Dashed Lines Correspond ro Elastic Response.

For a Z value .of 4 the ductility increases to 7.5 and the inelastic displacement exceeds the elastic by a factor of 1.87. For a Z of 8. the ductility increases 10 nearly 32 and the inelastic displacement becomes almost four times larger lhan the elastic. The curves in Fig. 18 iIIustrate lhis trend. Clearly, caution in specifying Z values is important in the short period range.

The definition of short period needs to be related 10 the predominant period of the ground. If one looks at Pan b of Fig. 18. the increasing trend in ductility demand for lhe Oakland Wharf record obtained during the 1989 Loma Prieta earthquake begins at a period near 1 second and even larger amplifications of ductility and displacement are observed for short period structures than predicted using the aboye equations. The amplified acceleration region of the spectrurn for this record is flat out to about 1 second. Thus. soil conditions are very irnportant in selecting the Z values as well.

Displacement histories (not shown) for the analytical model of the proof test specimen were nearly the same in the longitudinal direction as for the transverse direction. However, peak displacements were increased 30% as a result of the different dynamic characteristics of the model in this direcúon.



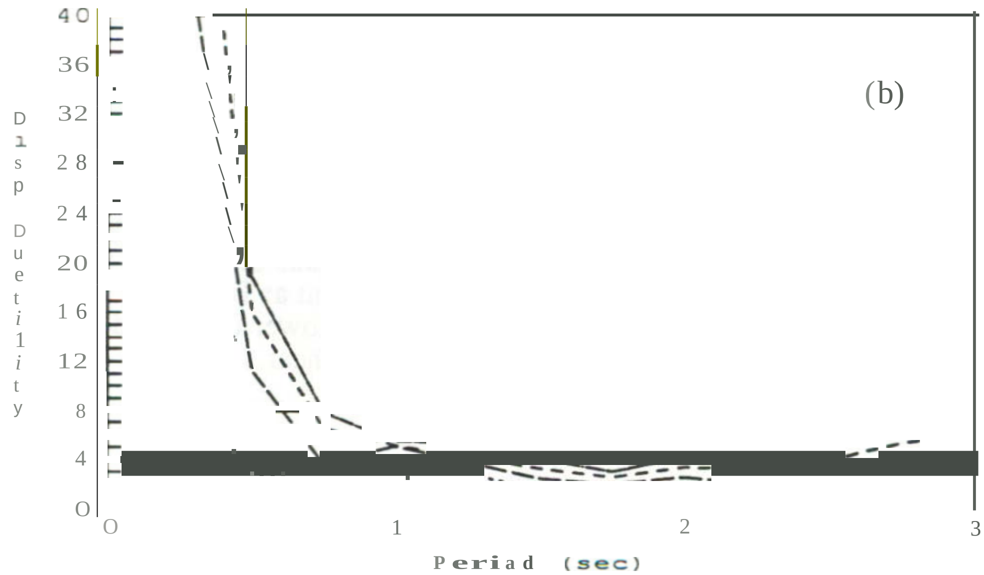


Fig. 18 Ductility Spectra for Constant Z. Values of Z equal to 2. 4. 6 and 8 are Included and Appear in Ascending Order.

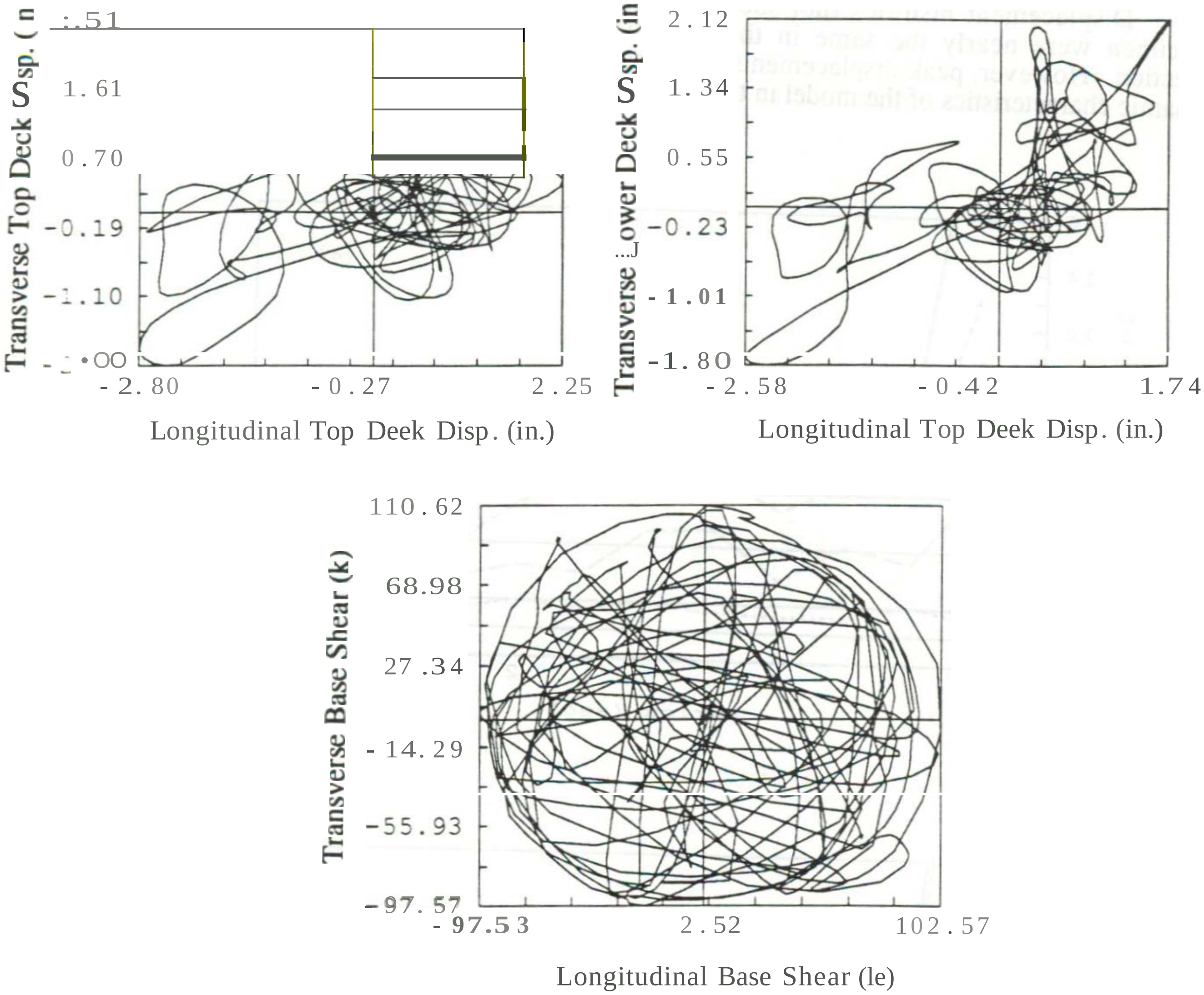


Fig. 20 Orbital Plots of Inelastie Response of Analytieal Model in Transverse Direetion to Two Horizontal Components of Taft Earthquake scaled to Z = 4;
(a) Transverse versus Longitudinal Displaeement at Top;
(b) Transverse versus Longitudinal Displaeement at Lower Deek; and
(e) Transverse versus Longitudinal Base Shear.

ACKNOWLEDGMENTS

The financial support of the California Department of Transportation is greatiy appreciated, In addition, the advice and guidance provided by J. Roberts, R. Zalinski, E. Thorkildsen and C. Whitten has been invaluable. The authors would also like to thank T. Jackson and J. Eddy of Parsons Brinkerhoff and R. Fenwick of the University of Canterbury for their help in interpreting the design drawings and calculations.

The able assistance of the Co-Principal Investigators on this project, Professors C. Thewalt and J. Moehle, is greatly appreciated. Numerous students have also been involved in this project, including I. Vasquez, S., Touli and F. Lam. The staff of the University of California is also greatiy appreciated in their efforts to facilitate the test. Staff members Dr. M. Blondett, W. McCracken, T. Merpole, L. Baker, R. Parsons and others helped immensely in carrying out the tests. Dr. A. Gilani and Dr. T. Yin provided considerable assistance in configuring and calibrating instrumentation.

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