PAPER NO. 561

# PROPOSED REVISIONS TO THE PAKISTAN HIGHWAY CODE FOR SEISMIC DESIGN OF REINFORCED CONCRETE BRIDGE PIERS

BY

ENGR. MUHAMMAD TARIQ AMIN CHAUDHRY

Engr. Muhammad Tariq Amin Chaudhary Paper No. 561

# PROPOSED REVISIONS TO THE PAKISTAN HIGHWAY CODE FOR SEISMIC DESIGN OF REINFORCED CONCRETE BRIDGE PIERS

Muhammad Tariq Amin Chaudhary [1] A.M. ASCE, M. IEPak, M. ACI, M. EERI, M. NZNSEE

## ABSTRACT:

Seismic activity in some regions of Pakistan is of such an intensity that it calls for a careful and detailed design of essential facilities and vital links like bridges to withstand these unpredictable loads. Pakistan Code of Practice for Highway Bridges, which was adopted in 1967, has serious deficiencies in the area of seismic design. In this paper a capacity design approach for the design of highway bridge piers is presented. Emphasis has been placed on determining the plastic collapse mechanism and the resulting moment capacity(s) of the hinges. Thereafter ductile detailing of beam-column joints and plastic hinge regions is carried out to ensure inelastic response without brittle shear failure. A brief historic overview on the performance of such structures is also given and modifications suggested to the Pakistan Highway Code (1967). Finally failure behavior of a two lane bridge pier is examined when designed for : (i) no seismic load, (ii) seismic load and detailing according to Pakistan Highway Code (1967) and (iii) designed and detailed according to the paper.

## 1. INTRODUCTION

The importance of bridges in today's integrated and interdependent society can not be overemphasized. These provide a safe and fast passage over natural and man-made obstructions. Besides serving the defense and strategic needs, these cater for the need of general public for transporting goods and passengers from one region to the other. Their significance increases manifold in case of an emergency caused by either natural disaster, war or an accident. Therefore their design, maintenance and operation is gaining significance as a result of valuable lessons learned from bridge failures and the resulting loss of life, property and business.

---

[1] Senior Engineer, Highway Division, National Engineering Services Pakistan (NESPAK), Pvt. Ltd.

Bridges are such an important component of the transportation network that these are supposed to be functional when the "need" arises i-e. during a natural disaster like hurricane, floods or earthquake. Despite this fact, bridges have almost invariably failed in the most spectacular fashion during floods and earthquakes. This can be attributed to :

a) Failure to estimate the design flood or earthquake

b) Failure to 'detail' the structure properly

c) A false sense of security on part of the designer in ignoring the occurrence of such a disaster. This is particularly true of earthquake loading which is taken too lightly by most of engineers (young ones in particular) due to non-recurrence of a seismic event in living memory.

## 2. PERFORMANCE OF BRIDGES DURING PAST EARTHQUAKES

Bridge failure during an earthquake can be classified into three categories (or combination thereof):

### a) Failure of the Super Structure

Owing to the stochastic and random nature of the earthquake excitation, the bridge as a whole can experience either a predominantly longitudinal or transverse forces. In case of excessive and differential movement of the super-structure with respect to the supporting sub-structure, the deck can displace or eventually fall from the bearing seats, thus causing complete failure of the basic purpose of the bridge. Due to its high stiffness and rigidity, the superstructure undergoes rigid body sort of movements and stresses in the various superstructure components remain with controllable limits. Failures have mostly resulted from insufficient bearing seat lengths or failure of anchoring devices. Steel rocker and roller type bearings have also performed poorly during earthquakes. These offer little or no energy dissipation and hence result in failure of bearing. These are being replaced with elastomeric (PTFE) bearings or base isolated devices. Continuity of superstructure is a desirable feature of earthquake resistant design of bridges. Retrofit measures to achieve this include the provision of positive cable connections between the superstructure and the supporting substructure, longitudinal joint restrainers, transverse bearing restrainers and vertical motion restrainers. Examples can be found in AASHTO 1992, Japanese Bridge Code (SEHB) 1980 and New York State Seismic Criteria for Bridge Rehabilitation, 1992.

### b) Failure of the Sub Structure

Substructure failure includes damage in the form of cracking / buckling of pier and abutment columns & plies and tilting / overturning of footings and wall type closed abutments. Pier and abutment transoms have remained undamaged in the recent earthquakes. Some of the worst failures of bridges have been attributed to this category. This happens due to designer's inability to foresee the possible collapse mechanism and subsequently poor attention to detail the energy dissipating plastic hinge zones. This makes the column susceptible to four modes of failure viz. pill out of main reinforcement, splice failures in the main reinforcement, brittle failure in the undesirable shear mode and loss of flexural capacity due to insufficient confinement.

### c) Failure of the Supporting Soil

If the supporting soil is vulnerable to earthquake excitations and tend to liquefy at a certain frequency (this is the case with saturated sandy soils), then the loss of bearing capacity can result in excessive vertical and/or horizontal movements which can result in severe structural damage to the bridge or can cause overturning /tilting of the substructure. On soft soils, certain modes of earthquake frequency contents can be drastically amplified, thereby increasing the severity of earthquake at a particular site with dire consequences. Therefore careful geotechnical investigations at important bridge sites should be carried out to preclude a disaster.

## 3. SEISMIC ACTIVITY IN PAKISTAN

Northern and western regions of Pakistan are seismically alive and had served as the epicenter of a number of severe earthquakes. Seismic activity in these areas is historically proven and evident from the geological features. Quoting Agha and Ahmed (1986) : *"There is evidence of active faults in several regions of Pakistan. Some of them are in the vicinity of large population centers. Pakistan is surrounded by a mountain system including the Himalayan and Baluchistan Arc which are well known for their seismic activity. The Himalayan faults have been associated with some of the largest seismic events with four events of Magnitude 8. Similarly Baluchistan and Hindukush mountains are characterized by several active faults including the well*

*known Chaman fault, which has caused movement in recent deposits at several locations along its trace. Hindukush mountains experience an average of 20 shocks per day which indicate the high intensity of seismic activity in these areas.*" Southern part of the country containing the important cities of Karachi and Hyderabad is also prone to earthquakes. Alla Bund fault, originating from the Rann of Katch and running in the vicinity of Karachi, is capable of triggering a magnitude 5 to 6 (MMI between VII and VIII) earthquake (Loya and Siddiqui 1986). Similarly, jolts of earthquakes are felt not too infrequently in the central and northern plains of Punjab. For detailed study of this subject refer Huda (1967), Ambraseys (1968), Ahmad (1974), Farah & DeJong (1979) and Adhami (1980).

## 4. NEED FOR SEISMIC DESIGN OF HIGHWAY BRIDGES

With the expansion of highway network in the country and commissioning of the first limited access Motorway in the past couple of years, a great number of bridges are being built throughout the country these days. The most often used bridge type is reinforced or prestressed concrete beams and slab arrangement. Usually the span for such type of arrangement is under 50m. For longer spans reinforced/prestressed box girders have also been employed.

West Pakistan Code of Practice for Highway Bridges was adopted in 1967 and contain very rudimentary provisions for seismic design of bridge components. These provisions are insufficient and misleading. These might have been the sate-of-the-art 30 year ago but now these should be replaced with better provisions based on the knowledge gained in the science and art of seismic design of bridges. Guidelines adopted in Japan, New Zealand and U.S.A. can be modified to suit local conditions and practices. The 1967 Pakistan Highway Code has the following provisions for earthquake resistant design:

2.17- Earthquake Stresses

In regions where earthquakes may be anticipated, provisions shall be made to accommodate lateral forces from earthquakes as follows:

$$EQ = CD$$

where

$EQ$ = Lateral force applied horizontally in any direction at center of gravity of the weight of the structure.

$D$ = Dead load of structure

$C$ = 0.02 for structure founded on spread footings on material rated as 4 tons of more per square foot.

= 0.04 for structures founded on spread footings on material rated as less than 4 tons per square foot.
= 0.06 for structures founded on plies.
Live load may be neglected.

The biggest drawback in these provisions is that seismic activity in a particular region, structural configuration and rigidity of the sub-structure and time period of the system have not been given any consideration while arriving at values of '*C*'.

## 5. PROPOSED PROVISIONS FOR SEISMIC DESIGN

Preparation of the Seismic Zoning map of Pakistan (Fig. 1) by NESPAK, which was ultimately adopted by the Housing and Physical Planning Department of the Government of Pakistan for the Pakistan Building Code (1986), is a milestone in imparting awareness about earthquake engineering among the engineering community of the country. This map has been used to arrive at some guidelines for seismic design of bridges in Pakistan.. Using the seismic zoning of the Pakistan Building Code (1986) and provisions of AASHTO (1992), the following guidelines are suggested:

### 5.1 Seismic Zoning Map And Acceleration Coefficients

Seismic Zoning map of the Pakistan Building Code (1986) has been adopted. Suggested acceleration Coefficients *A*, for each Zone are given in Table 1.

Acceleration coefficient for each zone is based on observation of Ahmad, 1990 and Uniform Building Code 1991. Higher Coefficient should be used near known active faults based on the seismic-tectonic study.

### 5.2 Methods Of Dynamic Analysis

There are three well known methods through which effects of seismic loads can be analyzed on a structure. The methods arranged in increasing degree of refinement, sophistication and cost are:

i) Equivalent Static Lateral Load Procedure
ii) Modal Analysis using Response Spectrum
iii) Time History Analysis

i) **Equivalent Static Lateral Load Procedure** is the simplest form of taking into account the effects of earthquake generated forces. In this procedure major importance is given to response in the fundamental (or first) mode of vibration and effects of higher modes is taken into account only indirectly. This is to be emphasized that this procedure gives satisfactory results for the slab-on-girder type bridge configuration having well balanced spans and supported by piers of equal stiffness (Roberts and Kershaw ,1984). As these type of bridges are mostly in use in Pakistan , so the above mentioned procedure can be reliably used for simple span bridges of upto 30 -40m span.

ii) **Modal analysis using response spectrum** entails the calculation of time periods of a chosen number of degrees of freedom after performing an eigenvalue analysis, finding the modal participation factor for each mode and then combining the resulting forces of each mode through SRSS (Square Root Sum of Squares) rule. For bridges with significantly irregular configuration or support stiffness, earthquake forces on different components should be determined by a more rigorous modal analysis. One such method, in a rather approximate and simple form is the single / multi-mode spectral analysis methods. Guidelines for this procedure are available in AASHTO 1992.

iii) **Time History Method** is the most sophisticated technique to analyze the effects of dynamic excitations caused by earthquake. Its widespread use in recent years is largely due to advancement in computer science, availability of fast machines, comprehension of structural and material behavior through laboratory testing of individual components and whole systems. As this is a very specialized field, therefore it should be used by experienced engineers only who have full understanding of the structural response and limitations inherent in the computer simulation packages.

As pointed out earlier that for the most common slab-on-girder type bridges, lateral load method gives satisfactory results, therefore in this paper only this technique will be elaborated for ease of use and simplicity of comprehension. For bridges of irregular configuration, method (ii) or (iii) should be used for seismic analysis.

## 5.3 Equivalent Lateral Seismic Force

Based on NEHRP-85, UBC-91 and AASHTO 1992 provisions, the following equation gives the Equivalent Lateral Seismic force:

$$EQ = C_E * D/R \qquad (1)$$

where CE is the Spectral acceleration found from design spectra and is given by

$$C_E = 1.2\,AS/T^{2/3} \leq 2.5\,A \qquad (2)$$

$R$ = Response Modification Factor, $D$ = Contributing dead wt of the structure $S$ = Soil factor and $T$ = Natural period of vibration in seconds.

Elastic Response spectrum describes the maximum response of a damped single-degree-of-freedom oscillator at various frequencies or periods. Spectral acceleration is obtained by normalizing the resulting acceleration response by peak ground acceleration. Response spectrum adopted for design purpose represents a smoothened envelope of such response spectrums subjected to a number of recorded / artificial earthquake time histories. Fig. 2 presents the elastic response spectrums adopted by various codes. It is obvious from this figure that for low period systems maximum value of $C_E$ applies. The maximum value of this coefficient divided by the Response Modification Factor, $R$ is given in Table 2 for various structural configurations of the substructure in the four seismic zones of Pakistan. Values of $R$ and $S$ are adopted from AASHTO 1992. Comparing these values with the Pakistan Highway Code 1967, it is evident that the code under-estimate effects of seismicity in all four zones which can simply spell disaster even in case of a moderate level seismic event.

In order to account for the directional uncertainty of earthquake motions, the seismic forces resulting from longitudinal and transverse loading conditions should be combined as follows:

**Load Case 1:** 100% Transverse Earthquake Actions + 30% Longitudinal Earthquake Actions

**Load Case 2:** 100% Longitudinal Earthquake Actions + 30% Transverse Earthquake Actions

Earthquake loads should be load factored and combined with other loads according to Table 3.22.1A of AASHTO, 1992.

# 6- DESIGN PROCEDURE

## 6.1 Philosophies of Seismic Design

Three main approaches may be used to proportion structural members to resist earthquake forces viz. Strength design, Direct Ductility design and Capacity design.

### a) Strength Design

In the strength design, all of the seismic force is resisted elastically by the structural components. This results in onerous members with unduly high cost of construction. This procedure is recommended only for important structures like nuclear containment structures.

### b) Direct Ductility design

Direct Ductility design procedure calls for detailing the members without any regard to the global behavior of the structure. Therefore, plastic hinges can form at undesirable locations which can cause large deflections causing stability failure or make the repair work costly.

### c) Capacity Design

The essence of capacity design philosophy is " identification of the weakest possible link in the structural system and ensuring that this link fails in the pre-determined and pre-destined ductile fashion". Capacity design procedures have become quite common in Japan, Europe, New Zealand and USA due to the decades long research at various institutions. In the capacity design process, dependable inelastic response is assured by the careful selection and detailing of plastic hinge locations which should be accessible and easy to repair. Inelastic deformations through flexural response are designed to take place in these regions only whereas other parts of the structure are designed to remain elastic. Non-ductile modes of inelastic deformation, such as shear failure are inhibited by first determining the ultimate strength of these regions taking into account all sources of overstrength (like unintentional increase of steel yield strength, strain hardening or increase in $f_c'$) and then designing the remainder of the structure to resist forces consistent with the ultimate strength of these

regions. (Paulay and Priestley, 1992). This philosophy will be demonstrated through a design example later in this section.

## 6.2 Ductility Demand Vs Ductility Capacity

Both wind load and earthquake load are lateral in nature but structure behaves in entirely different fashion when subjected to them. It is generally sufficient to provide adequate strength and stiffness while designing for wind loads but in the case of seismic design a third basic requirement, that of ductility must also be considered. This difference is due to the fact that wind load is resisted entirely through elastic action and there is no excursion into the inelastic behavior. While on the other hand, resisting seismic loads through elastic response results in a very costly structure, therefore almost all structures are designed to resist seismic load through some degree of inelastic action. This is only possible if the critical components of the structure are properly designed to undergo inelastic rotations and displacements. Ability of a structure to undergo inelastic deformations is termed "ductility". For instance, displacement ductility is defined as the ratio of the maximum inelastic displacement to the displacement at first yield.

Ductility demand in a structure can be fully met only if the structure is designed not only for strength and stiffness considerations but also properly detailed to preclude brittle failure associated with shear and bond / anchorage. Response reduction factor, 'R' defined earlier is nothing but a rough estimate of ductility of a structural system. In order to achieve this minimum level of ductility, provisions described in later sections should be strictly followed.

## 6.3 The 'Art' of Seismic Detailing

Fig. 3 depicts the contrasting differences between ductile and non-ductile detailing of a typical bridge pier in a qualitative manner. Quantitative calculations and code requirements for different parts are presented in the next section under pertinent headings.

## 6.4 Design Example

The design process will be clarified through an example. The example bridge is shown in Fig. 4 . The design process is shown in Fig. 5.

The process is self-explanatory upto the calculation of design actions after determining the equivalent lateral seismic force as outlined above and thereafter performing manual or computer aided structural analysis.

## a) Design of Longitudinal Reinforcement

Design of longitudinal reinforcement for seismic loads differ from its static counter part in the value of under capacity factor ($\phi$) for columns. The relationship for $\phi$ according to AASHTO (1992) is

$$\phi = 0.9 - 2(P/f_c'A_g) \geq 0.5 \qquad (3)$$

This stringent reduction factor for higher $P/f_c'.A_g$ values is to account for the loss of ductility associated with heavily loaded columns.

## b) Shear Design

Current seismic design philosophy is to ensure against failure by setting the shear strength of a bridge pier higher than the maximum flexural strength that can be developed. The design shear forces are thus not directly related to the design horizontal inertia loading, but depend on maximum flexural strength that could actually be developed in the plastic hinges. These actual shear forces generated during an earthquake may be as high as three times the code specified nominal values (Verma and Priestley, 1994). Therefore, it is essential to determine the capacities at each hinge location and the collapse load required for a plastic mechanism. The following sub-section describes a convenient method of determining these capacities.

### Plastic Collapse Mechanism and Overstrength Capacities

Fig. 6 depicts Plastic Collapse Mechanism for the bent for transverse direction earthquake with plastic hinges forming in the columns. Plastic moment capacity at each hinge location is found from the column interaction diagram using the usual ACI 318 assumptions viz. an ultimate compression strain $\epsilon_{cu} = 0.003$, an average concrete stress across the stress block of $0.85f_c'$ with depth of $a=\beta_1 c$ where $c$= neutral axis depth and $\beta_1 = 0.85$ for $f_c' = 3000$ psi. Overstrength moments are calculated for $\phi=1$ for both flexure and axial load i-e. no undercapacity factor is used Employing

the virtual work method of plastic analysis:

$$F = \frac{\sum_{i=1}^{4} M_i}{L_c} \quad (4)$$

where $M_i$ is the plastic hinge moment capacity determined as outlined above and $L_c$ is the clear height of column. Determination of $F$ and $M_i$ in Eq. (4) is of an iterative nature as $M_i$ is a function of axial load. The changes in the axial load are found by determining overturning reactions ($T$ & $C$) by taking moments about the base:

$$F * L_c = M_1 + M_2 + T * L_b$$

$$T=C= (F.L_c - M_1 - M_2) / L_b \quad (5)$$

These overturning reactions are added ($C$) or subtracted ($T$) from the gravity reactions and the moment capacity at each hinge is found iteratively solving Eq. (4) and (5).

Based on these overstrength moments, column shear is determined such that

$$V = (M_1 + M_2) / L_c \quad (6)$$

and hoops provided accordingly.

## c) Detailing of Plastic Hinge Locations

Plastic hinge locations are detailed to provide sufficient rotational ductility to withstand inelastic movements. Preferably, such locations should be easily accessible for ease of repair/retrofit after an earthquake. Length of such confinement regions as given by AASHTO 1992 is largest of : maximum cross sectional dimension, 1/6 clear column height or 450 mm.

Confining steel, in the form of hoops, for rectangular columns should be

$$A_{sh} = 0.3\, s\, h_{core}\, (A_g/A_c - 1)\, f_c'/f_y \quad (7)$$

and spacing of transverse hoops according to AASHTO 1992 shall be less than or equal to 75mm, while this spacing according to NZS 3101 (1982) and CALTRANS (1990) should be

$$s \le 6d_{bl} \le D/5 \le 200\ mm \quad (8)$$

where $d_{bl}$ = longitudinal bar diameter and $D$ = gross cross -sectional dimension of pier.

## d) Joint Reinforcement

No or very little attention is paid to the proper design and detailing of beam-column joints. It is now well understood and well documented that poor performance of these regions have critically damaged the bridge sub-structure. Preferably joint should not undergo inelastic deformations for two main reasons: firstly these regions possess poor energy dissipating characteristics and any inelastic behavior in these regions contribute to significant deformations of the entire structural system thereby generating large P-Δ moments which can cause global stability problems and secondly accessibility and post-earthquake repair of these locations is extremely tedious if not impossible. This elastic behavior is ensured while designing according to capacity design principles. Forces acting on a typical column-transom joint are shown in Fig. 7(a). Both column shear ($V_{col}$) and joint shear ($V_{jh}$) are calculated on the basis of overstrength moment capacity for reasons explained earlier. From equilibrium conditions of the joint shown in Fig.7(a)

$$T = M_p/jd \quad (9)$$

$$\text{and } V_{jh} = T - V_{col} \quad (10)$$

There is not a consensus among various codes on the design procedure for beam-column joints for seismic resistance and wide difference exist (Leon, 1990). Requirements of some of these codes are as follows:

### AASHTO 1992

AASHTO only require, without any calculations, to simply extend the shear reinforcement in confinement zones to one half the joint depth.

### ASCE ACI 352 and CAN3-A23

For rectangular columns

$$A_{sh} = 0.3\, s\, h''\, f_c'/f_y(A_g/A_c - 1) \quad (11)$$

but not less than

$$A_{sh} = 0.09\, s\, h''\, f_c'/f_y \quad (12)$$

where $A_{sh}$ = total cross sectional area of transverse steel ($in^2$), s= spacing of reinforcement (in), h" = confined cross sectional dimensions measured center to center of confining steel (in) and $f_{yh}$ = yield stress of confining steel (psi). It is also required that concrete stress in the joint should not exceed $12\, f_c'^{1/2}$ (psi).

### New Zealand Code NZS 3101 (1982)

The recommendations of New Zealand code NZS 3101 (1982) are based on the analytical and experimental work done on full scale beam-column joints by Paulay et al (1978). The shear transferring mechanism in a joint is divided into two parts viz. concrete strut and panel truss mechanisms as shown in Fig. 7(b). The diagonal strut mechanism is formed along the main diagonal of the joint panel caused by compression and shear on concrete acting at the joint boundary. Whereas truss mechanism is formed by bond stress transfer along the column and longitudinal reinforcement, tensile resistance of lateral reinforcement and compression resistance of uniform diagonal concrete struts in the joint. It should be noted that lateral reinforcement in joint enhances the truss mechanism only with no effect on the strut mechanism. (Paulay and Priestley 1992)

Joint shear carried by concrete through strut mechanism $V_{ch}$, is given by

$$V_{ch} = 0.3\,(1 + 3.5\,P / f_c' A_g) V_{jh} \qquad (13)$$

and the remaining shear has to be carried by joint reinforcement such that

$$V_{sh} = \phi\, A_s h\, f_y \qquad (14)$$

Complete detailing of the example bridge pier is shown in Fig. 8.

## 7. COMPARISON OF DESIGNS

In this section a comparison of the ductility capacity is made for 3 designs. Its purpose is to examine the performance of each design under earthquake loading and categorize their failure behavior according to procedure of ATC 6-2 (1983). The following designs are considered:

i) Gravity Loads only
ii) Seismic Loading according to Pakistan Highway Code (1967)
iii) Seismic Loading according to procedure described in the paper

Load combinations for ultimate strength design are considered according to AASHTO 1992 for all the three designs but seismic detailing procedures as outlined in the paper are only adopted for design (iii). Design actions and reinforcement for the three cases are tabulated in Table 3.

## 7.1 Strength Degradation Model

Procedure recommended by ATC 6-2 (1983) is used to assess the ductility of the three designs for increasing drift levels. The procedure can be summarized as follows:

Step 1: Find the shear corresponding to the flexure mechanism ($V_{flexure}$)

Step 2: Find the initial shear strength ($V_i$)

Step 3: Find the final shear strength ($V_f$)

Step 4: Classify the failure behavior as follows (See Fig. 9)

$$\left.\begin{array}{lll} V_{flexure} < V_f & & \text{Ductile} \\ V_f < V_{flexure} < V_i & & \text{Moderate Ductility} \\ V_i < V_{flexure} & & \text{Brittle} \end{array}\right\} \quad (15)$$

For the Moderate ductility case, maximum displacement ductility value, $\mu_\Delta$, is set at 5 after which $V_f$ applies. Displacement ductility ($\mu_\Delta$) is defined as

$$\mu_\Delta = \Delta_{max}/\Delta_y \quad (16)$$

where $\Delta_{max}$ is the maximum displacement and $\Delta_y$ is displacement at first yield.

$\mu_\Delta = 5$ can be taken for structures not designed and detailed according to seismic procedure. If seismic design and detailing practice is followed then $\mu_\Delta$ should be calculated according to the following equation proposed by Priestley and Park (1987):

$$\mu_\Delta = 1 + 3(\phi_{max}/\phi_y - 1)(1 - 0.5\, L_p/L)\, L_p/L \quad (17)$$

where $\phi_{max}$ = ultimate curvature, $\phi_y$ = curvature at first yield, $L_p$ = equivalent plastic hinge length = 0.5 * length of plastic hinge region and $L$ = length of column between points of maximum and zero moment.

It is to be noted that ATC 6-2 (1983) procedure is a bit conservative and better energy based strength degradation models are now available (Mander et al , 1993). But due to its simplicity and statuary authority, ATC 6-2 (1983) procedure has been adopted in this paper.

## 7.2 Shear Strength at Various Stages

Shear strength corresponding to flexural mechanism ($V_{flexure}$) for each column is given by

$$V_{flexure} = 0.5\,F \qquad (18)$$

where $F$ is given by Eq.(4)

Calculation of Initial and final Shear strength is carried out by using the improved equations of Priestley et al (1994) such that

$$V_i = V_{ci} + V_s \qquad (19)$$

$$\text{and } V_f = V_{cf} + V_s \qquad (20)$$

where $V_{ci} = 3.5\,(f_c')^{1/2} A_e$ in psi units. $A_c$ is the effective shear area given by $0.8A_g$ and $V_{cf} = 1.2\,(f_c')^{1/2} A_e$ for seismically detailed column and $V_{cf} = 0$ otherwise. Shear carried by steel $Vs$ is given by

$$V_s = A_v f_y D'/s \qquad (21)$$

where $D'$ is the center to center distance of hoops.

## 7.3 Results

Numerical values of these quantities for the three cases are given in Table 4 and plotted in Fig. 10. Following observations can be made:

**i)** Failure behavior of cases (i) & (ii) can be classified as moderately ductile while case (iii) can be termed as ductile according to the ATC 6-2 classification. It is obvious from Fig. 10 that cases (i) & (ii) are not capable of resisting large deformations associated with seismic excitations and reduction in shear strength with increasing displacement is substantial. On the other hand case (iii) which was designed and detailed according to the seismic guidelines possesses enough ductility such that there is no loss of shear strength with increasing displacement.

**ii)** It can be noted that design and behavior of design (i) and (ii) is similar i-e. designing for seismic load suggested by Pakistan Highway Code (1967) is as good as neglecting this load altogether.

**iii)** Despite being seismically deficient the failure behavior of design (i) and (ii) is classified as moderately ductile which is mainly due to low $M/f_c' A_g h$ ratio

and longitudinal steel requirement based on gross cross-sectional area ($\rho_{min}=0.01A_g$). As aesthetic considerations are still rated very low on a bridge designer objectives, therefore increased dimensions are a common practice in local design offices. These stocky members may be visually offensive and aesthetically displeasing but reserve strength is inherent in their design which can be mobilized during a moderate seismic event. But this should be kept in mind that this sort of practice does not guarantee a safe and dependable performance during severe earthquake.

**iv)** It is evident from the behavior of design (iii) that shear strength in excess of that required by the flexure mechanism is only a wastage. Therefore adding a lot of transverse steel without keeping in view the requirement of $V_{flex}$. will only drive the cost upwards without any performance enhancement.

## 8. CONCLUDING REMARKS

It has been shown that the current provisions of Pakistan Highway Code underestimates the forces generated by earthquakes in almost all regions of Pakistan. Therefore these should be updated as suggested in the paper. The second part of the paper emphasized the need for seismic detailing; which is currently non-existent in our design practice, to preclude brittle shear failure. This was demonstrated through comparison of three different designs and it was concluded that pier designed and detailed for seismic loading possess better ductility characteristics. This technique has given excellent results in the recent Northridge (Todd et al, 1994) and Kobe (Park et al, 1995) earthquakes. Seismic detailing is more of an art than a science and is not the name of putting a lot of steel but it stands for placing right amount of steel at the right locations. The extra cost of such detailing is peanuts when compared with the benefits accrued during a quake.

TABLE 1: Acceleration Coefficient 'A' for Seismic Zones of Pakistan

| ZONE | 'A' |
|---|---|
| 0 | 0.075 |
| 1 | 0.15 |
| 2 | 0.20 |
| 3 | 0.30 |

TABLE 2: Maximum Equivalent Lateral Force Coefficient ($C_E$/R) in Seismic Zones of Pakistan

| Type of Foundation | Zone 0 | Zone 1 | Zone 2 | Zone 3 |
|---|---|---|---|---|
| Wall type Pier | 0.09375 | 0.1875 | 0.25 | 0.375 |
| Reinforced Concrete Pile Bents | | | | |
| a. Vertical Piles Only | | | | |
| b. One or more Batter Piles | 0.0625 | 0.125 | 0.167 | 0.25 |
| | 0.09375 | 0.1875 | 0.25 | 0.375 |
| Single Columns | 0.0625 | 0.125 | 0.167 | 0.25 |
| Steel or Composite Steel and Concrete Pile Bents | | | | |
| a. Vertical Piles Only | 0.0375 | 0.075 | 0.10 | 0.15 |
| b. One or more Batter Piles | 0.0625 | 0.125 | 0.167 | 0.25 |
| Multiple Column Bent | 0.0375 | 0.075 | 0.10 | 0.15 |

TABLE 3: Comparison of Design Actions & Reinforcement for the Three Designs

| S.No. | Case | EQ (kN) | $P/f_c'A_g$ | $M/fc'A_gh$ | Long. Steel | Hoops |
|---|---|---|---|---|---|---|
| 1- | Gravity loads only | 0 | 0.317 | 0.003 | 12 # 8 (based on $\rho_{min}$) | $A_v = 260\ mm^2$ # 4 @ 300 Throughout |
| 2- | Seismic #1 (according to Pakistan Highway Code (1967)) | 66 | 0.135 | 0.027 | 12 # 8 (based on $\rho_{min}$) | $A_v = 260\ mm^2$ # 4 @ 300 Throughout |
| 3- | Seismic # 2 (according to the paper) | 333 | 0.118 | 0.107 | 12 # 9 | $A_v = 440\ mm^2$ #4 @ 75 in confinement zones #4 @ 225 elsewhere |

TABLE 4: Comparison of Shear Strength of the Three Designs (kN)

| S. No. | Case | $V_{flexure}$ | $V_{ci}$ | $V_{cf}$ | $V_c$ | Remarks |
|---|---|---|---|---|---|---|
| 1- | Gravity | 400 | 614 | 0 | 214 | Moderate Ductility |
| 2- | Seismic #1 | 400 | 614 | 0 | 214 | Moderate Ductility |
| 3- | Seismic #2 | 458 | 614 | 210 | 486 | Ductile |

## REFERENCES:

**AASHTO (1992),** "Standard Specifications for Highway Bridges", American Association of State Highway and Transportation Officials, Washington, D.C., 686pp.

**ACI Committee 318 (1989),** "Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary - ACI 318 R-89", American Concrete Institute, Detriot, 353 pp.

**ASCE-ACI Committee 352 (1985),** "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352 R-85)", American Concrete Institute, Detroit, 18 pp.

**ATC 6-2 (1983),** "Seismic Retrofitting Guidelines for Highway Bridges", Applied Technology Council, California, 220 pp

**Adhami, N.Z. (1980),** " Seismicity in Pakistan and its Relevance to Engineering Structures", The Pakistan Engineer, Vol.III, No.11&12, The Institution of Enginneers Pakistan, Lahore, pp.1-48.

**Agha, A.M. and Ahmed, M. (1986),** "Requirement and Application of Earthquake Engineering in Pakistan", Proceedings of National Conference on Earthquake Engineering, 28th Annual Session of the Institution of Engineers Pakistan, Quetta.

**Ahmad, M. (1990),** "Seismic Microzoning of Quetta", Seminar on improvement of Quetta, Quetta Development Authority, Quetta, pp. 8-1 to 8-5.

**Ahmad, M. (1974),** "The classification of Hazard Zones in Pakistan", Proceedings of CENTO Symposium on Earthquake Engineering and Earthquake Seismology, Middle East Technical University, Ankara.

**Ambraseys, N.N. (1968),** "A Note on the Seismicity of West Pakistan", Proceedings of CENTO Conference on Earthquake Hazard Mitigation, Ankara, Vol.1, p.89

**CALTRANS (1990),** " Standard Specifications for Highway Bridges Relating to Seismic Design", California Department of Tranportation, Division of Structures.

**CAN3-A23.3-M84 (1984),** "Design of Concrete Structures for Buildings", Canadian Standards Associations, Montreal, 281pp

**Code of Practice for Highway Bridges (1967)**, Govt. of West Pakistan, Highway Department

**Farah, A. and DeJong, K.A. (Editors) (1979)**, " Geodynamics of Pakistan", Geological Survey of Pakistan, Quetta, 361 pp.

**Huda, S.Q. (1967),** "Seismology in Pakistan", Individual studies by participants at the International Institute of Seismology and Earthquake Engineering, Tokyo, Vol.4, p.1

**Leon, R.T. (1990),** "Shear Strength and Hysteretic Behavior of Interior Beam-Column Joints", ACI Structural Journal, Vol.87, No.1, American Concrete Institute, Detroit, pp 3-11.

**Loya, A.R. and Siddiqui, K.M. (1986),** "Evaluation of Earthquake Forces and Design of Earthquake Resistant Buildings for Karachi". Proceedings of National Conference on Earthquake Engineering, 28th Annual Session of the Institution of Engineers Pakistan, Quetta.

**Mander, J.B., Waheed, S.M., Chaudhary, M.T.A., Chen, S.S. (1993),** " Seismic Performance of Shear-Critical Reinforced Concrete Bridge Piers", Technical Report NCEER-93-0010, Nattional Center for Earthquake Engineering Research, Buffalo, NY, 176pp.

**NEHRP-85 (1985),** " 1985 Edition of NEHRP Recommended Provisions for the Development of Seismic Regulations of New Buildings", Federal Emergency Management Agency, Washington, D.C.

**NZS 3101 (1982),** "Code of Practice for the Design of Concrete Structures", Standards Associations of New Zealand, Wellington, 127pp.

**New York State Seismic Criteria for Bridge Rehabilitation (1992),** New York State Department of Transportation, Albany, 23pp.

**Pakistan Building Code (1986),** Govt. of Pakistan, Ministry of Housing and Works, Environment and Urban Affairs Divisions, Islamabad.

**Park, R. and 12 others (1995),** " The Hyogo-Ken Nanbu (Kobe) Earthquake of 17 January 1995", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol.28, No.1, pp. 1-98.

**Paulay, T. and Priestley, M.N.J. (1992),** "Seismic Design of Reinforced Concrete and Masonry Buildings", John Wiley and Sons, New York.

**Paulay, T., Park, R. and Priestley, M.N.J. (1978),**" Reinforced Concrete Beam-Column Joints under Seismic Action", ACI Journal, Vol.75, No.11, American Concrete Institute, Detroit, pp 585-593.

**Priestley, M.N.J., Seible, F., Xiao, Y and Verma, R. (1994),** "Steel Jacket Retrofitting of Reinforced Concrete Bridge Columns for Enhanced Shear Strength - Part 1: Theoretical Considerations and Test Design", ACI Structural Journal, Vol.91, No.4, American Concrete Institute, Detroit, pp 394-405.

**Priestley, M.N.J. and Park, R. (1987),** "Strength and Ductility of Concrete Bridge Columns Under Seismic Loading", ACI Structural Journal, Vol.84, No.1, American Concrete Institute, Detroit, pp 61-76.

**Roberts, J.E. and Kershaw, R.E. (1984),** "Seismic Design for Sacramento Light Rail Transit System", Lifeline Earthquake Engineering: Performance, Design and Construction. Proceedings of a Symposium sponsored by TCLEE of ASCE, San Fransisco, U.S.A. pp. 37-51

**SEHB (1980),** "Specifications for Earthquake-Resistant Design of Highway Bridges", Japan Road Association, Tokyo.

**Todd, D. and 7 others (1994),** " 1994 Northridge Earthquake : Performance of Structures, Lifelines and Fire Protection Systems", NIST Special Publication 862, National Institute of Standards and Technology, Gaithersburgh, MD, 181pp.

**UBC-91 (1991),** " Uniform Building Code", International Conference of Building Officials, Whittier, California.

**Verma, R. and Priestley, M.N.J. (1994),** "Optimal Trends in Seismic Design of Single Column Circular Reinforced Concrete Bridge Piers", Earthquake Spectra, Vol.10, No.3, Earthquake Engineering Research Institute, California, pp 589-614.

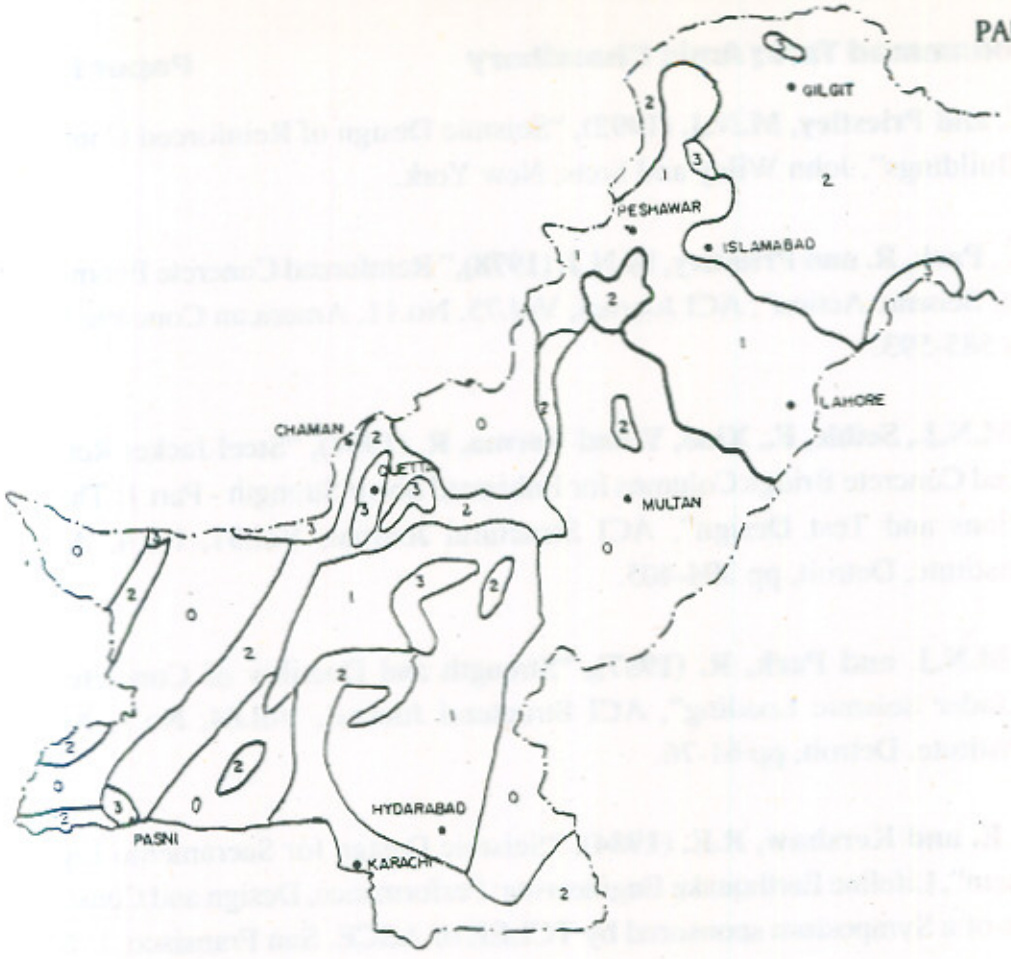



Fig. 1: Seismic Zoning Map of Pakistan (adopted from Pakistan Building Code, 1986)

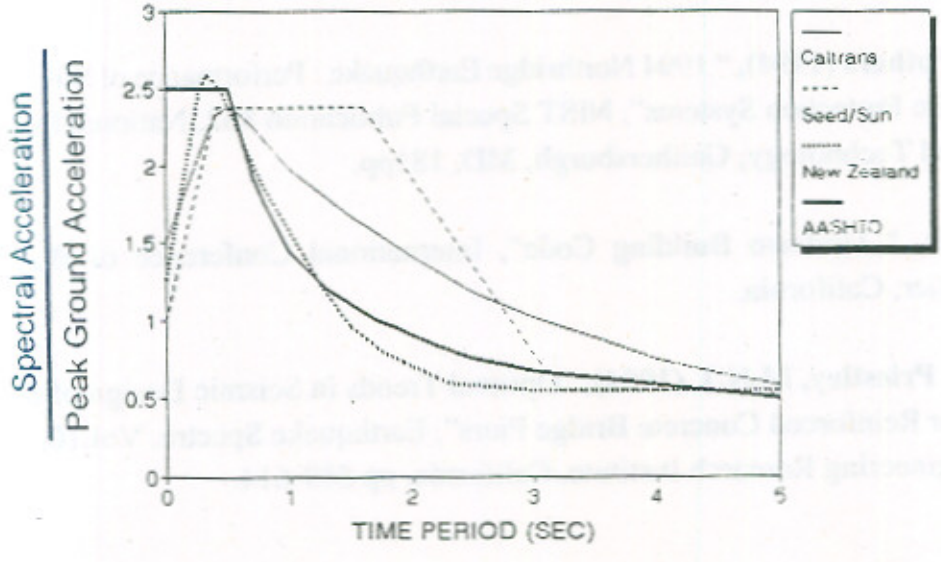



Fig. 2: Comparison of Elastic Design Spectra of Various Codes

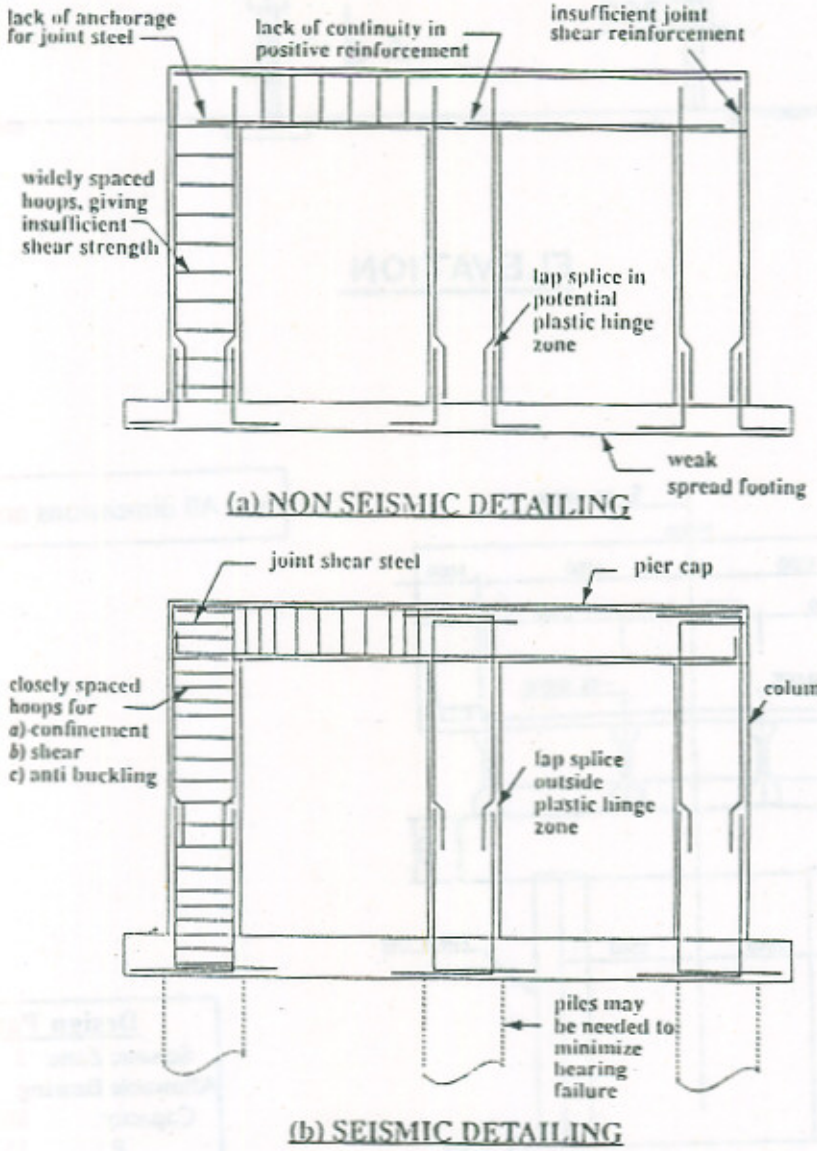
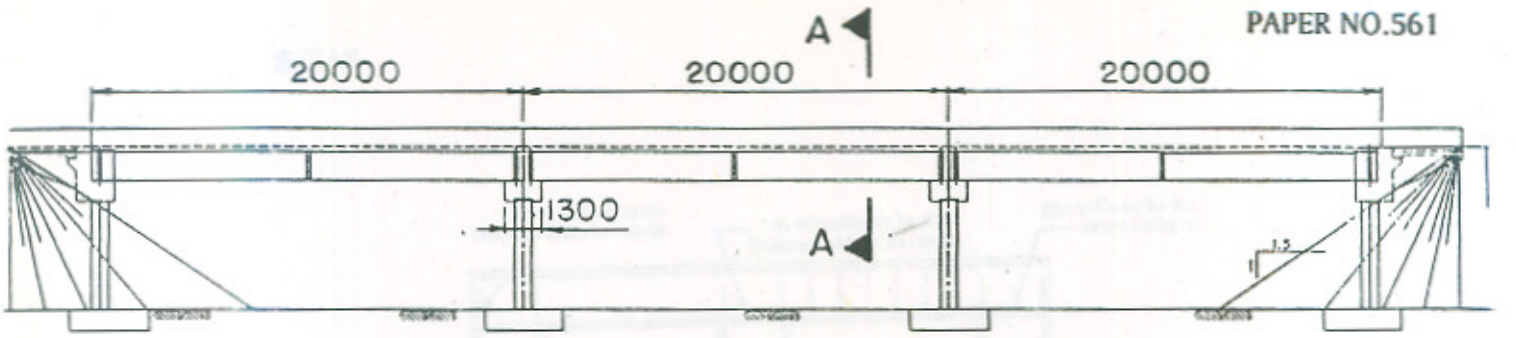
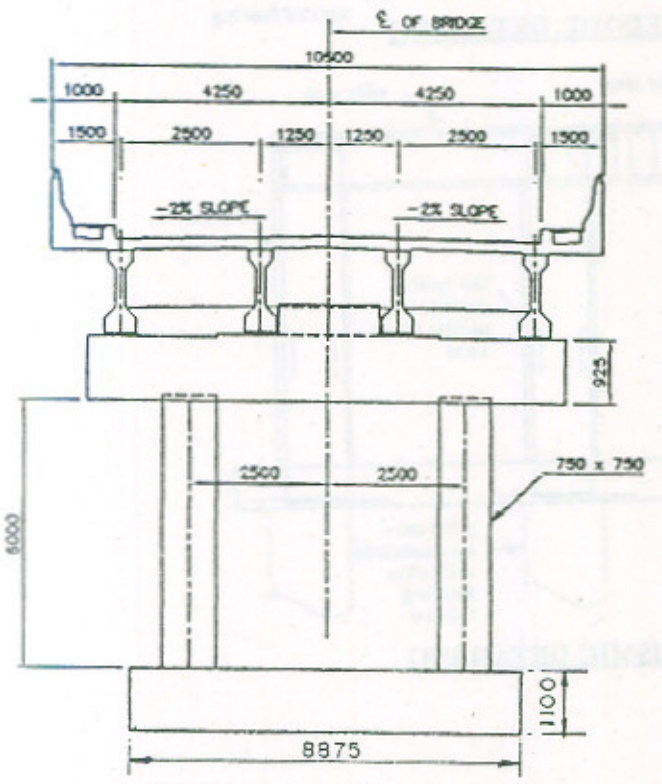



Fig. 3: Differences in the Reinforcement Detailing of Bridge Piers as Result of the Design Philosophy




ELEVATION




CROSS SECTION A–A

- All dimensions are in millimeters

**Design Parameters**

| | |
|---|---|
| Seismic Zone: | 2 |
| Allowable Bearing Capacity: | 80 T/m$^2$ |
| $P_a$: | 3333 kN |
| Time Period. T: | 0.52 sec. |
| $f_c'$: | 20.69 MPa |
| fy: | 417 MPa |
| Live Load: | Class 'A' or 'AA' of Pakistan Highway Code (1967) |

Fig. 4: Design Parameters and Layout of the 2-lane Example Bridge

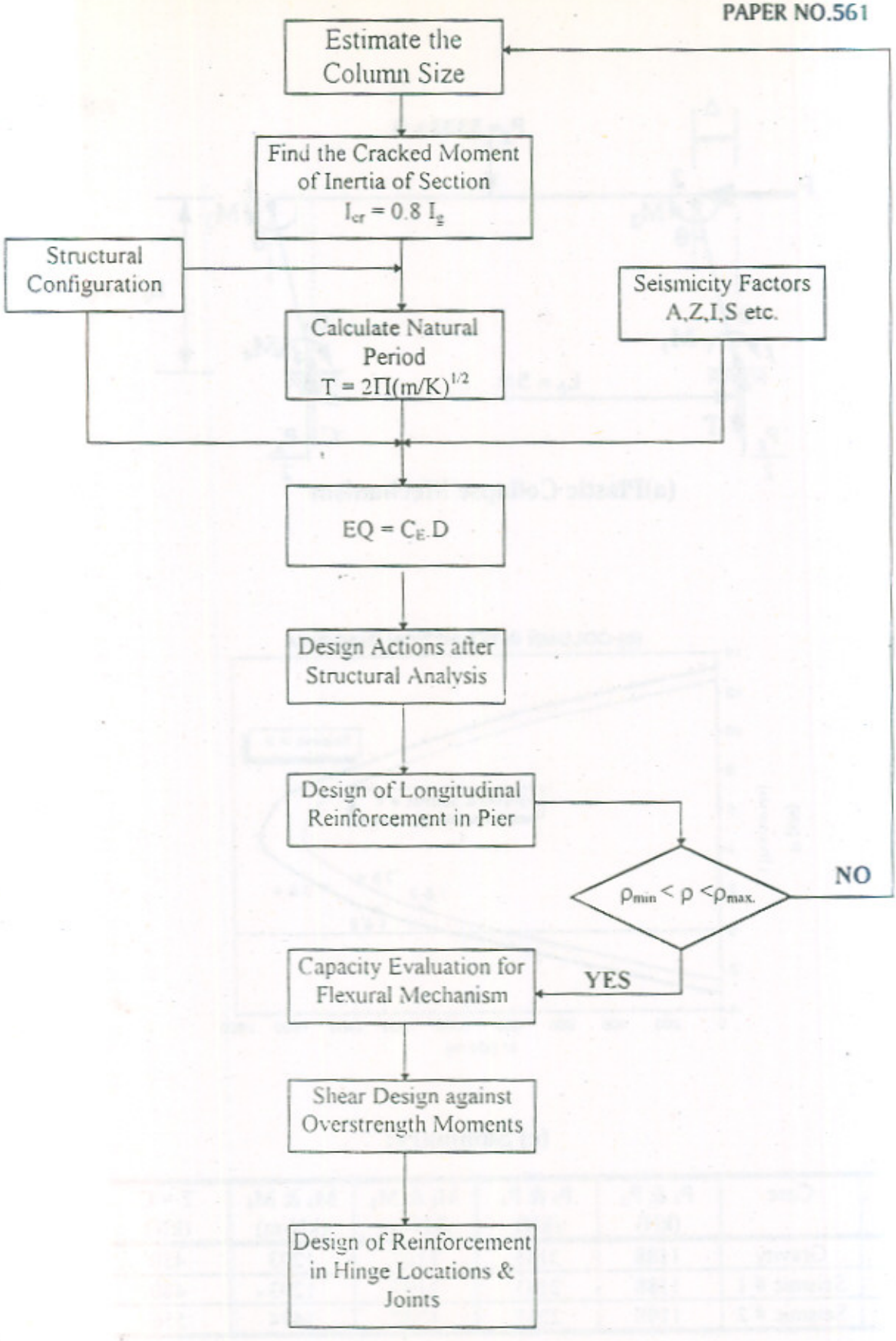
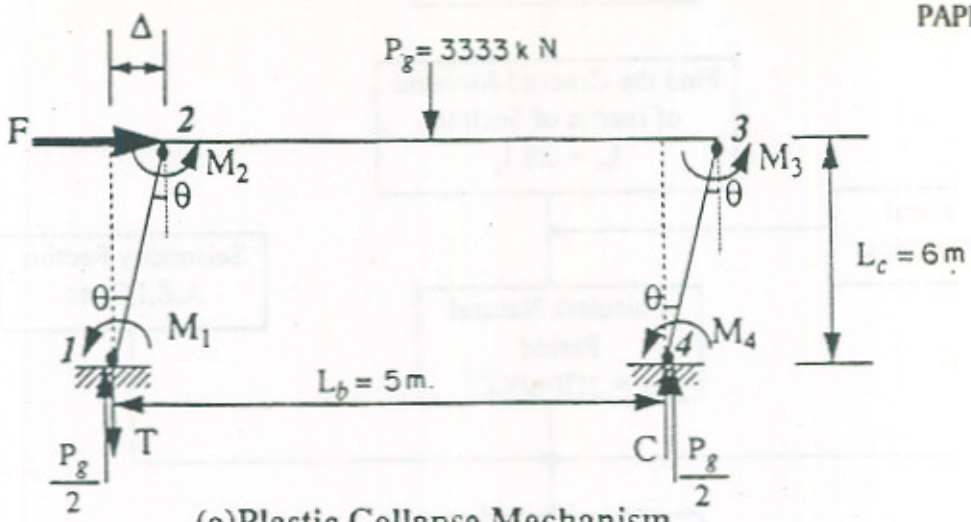
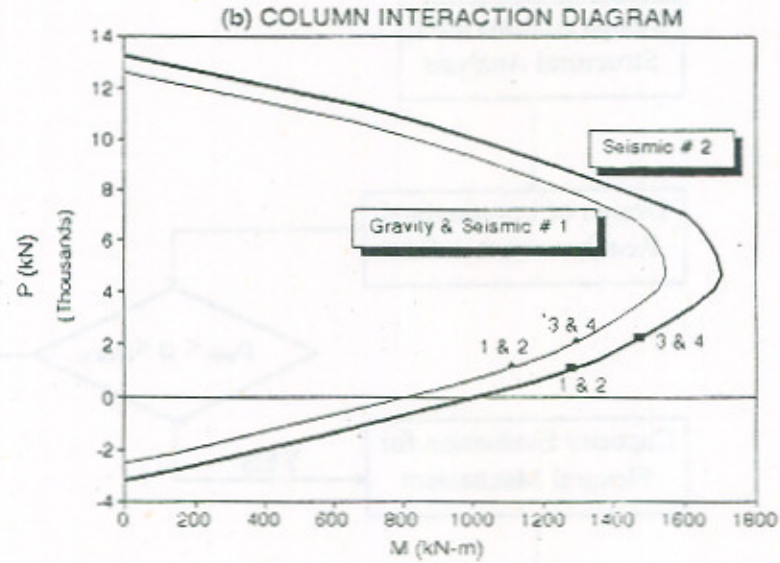



Fig. 5: Outline of Seismic Design Process for Reinforced Concrete Highway Bridge Piers




(a)Plastic Collapse Mechanism




(c) Summary:

| S.No. | Case | $P_1$ & $P_2$ (kN) | $P_3$ & $P_4$ (kN) | $M_1$ & $M_2$ (kN-m) | $M_3$ & $M_4$ (kN-m) | T = C (kN) | F (kN) |
|---|---|---|---|---|---|---|---|
| 1- | Gravity | 1188 | 2145 | 1107 | 1293 | 480 | 800 |
| 2- | Seismic # 1 | 1188 | 2145 | 1107 | 1293 | 480 | 800 |
| 3- | Seismic # 2 | 1108 | 2215 | 1267 | 1474 | 556 | 916 |

Fig. 6 : Plastic Collapse Mechanism and Determination of Collapse Load

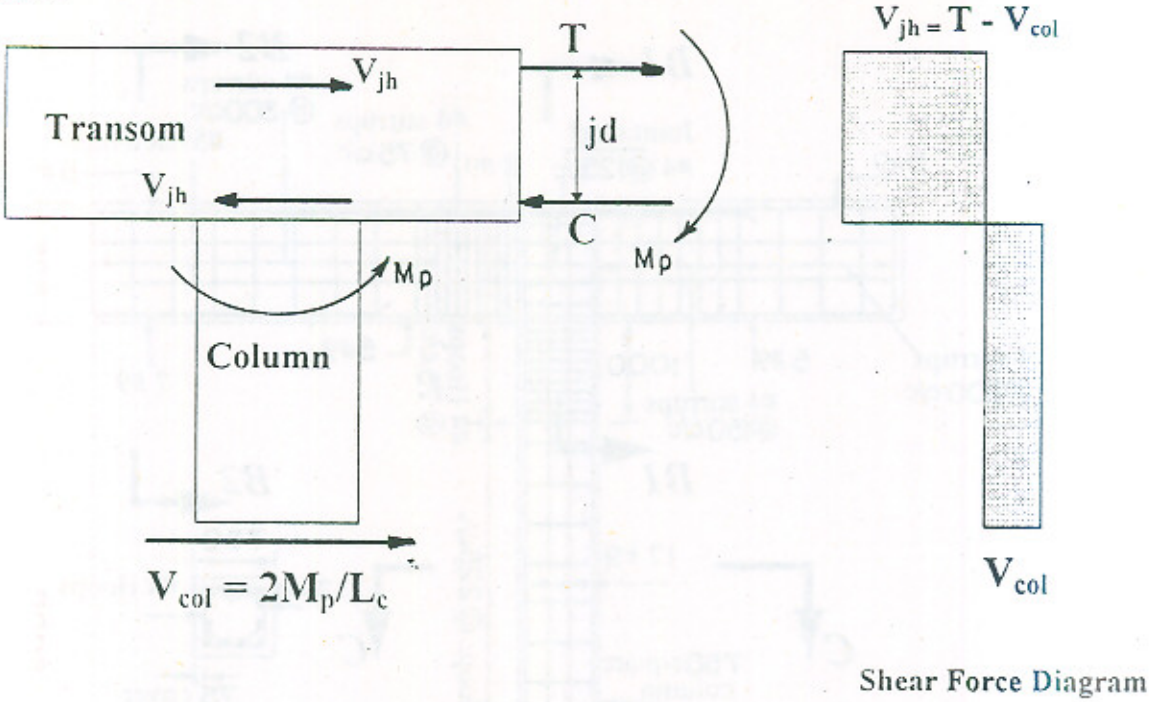



Fig. 7(a): Forces Acting on the Transom-Column Joint

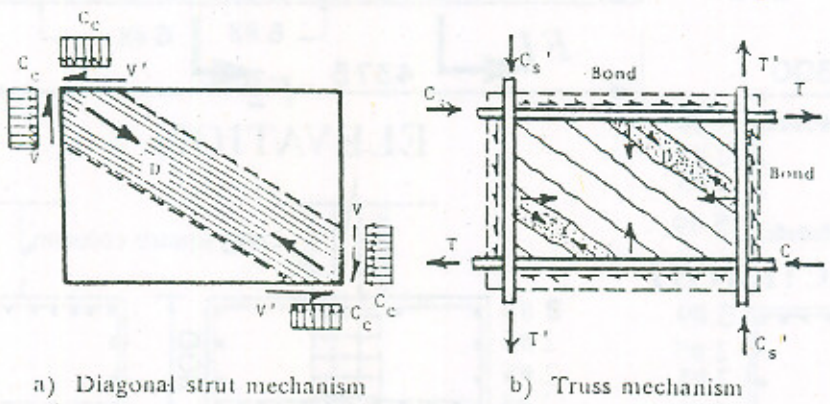
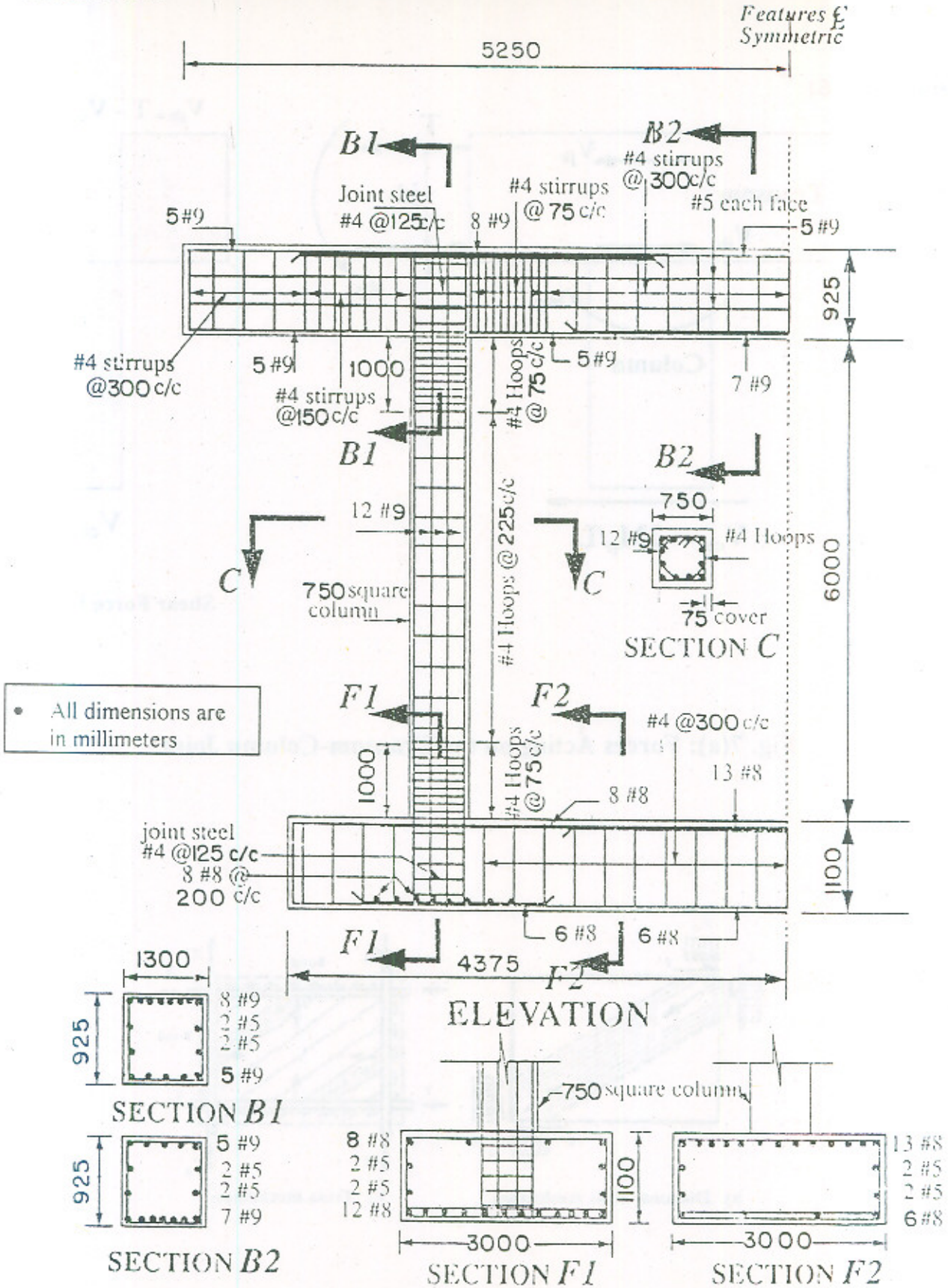



Fig. 7(b): Shear Resisting Mechanisms in a Transom-Column Joint
(adopted from Paulay and Priestley, 1992)




• All dimensions are in millimeters

**Fig. 8: Seismic Detailing of Reinforcement for the Example Bridge Pier**

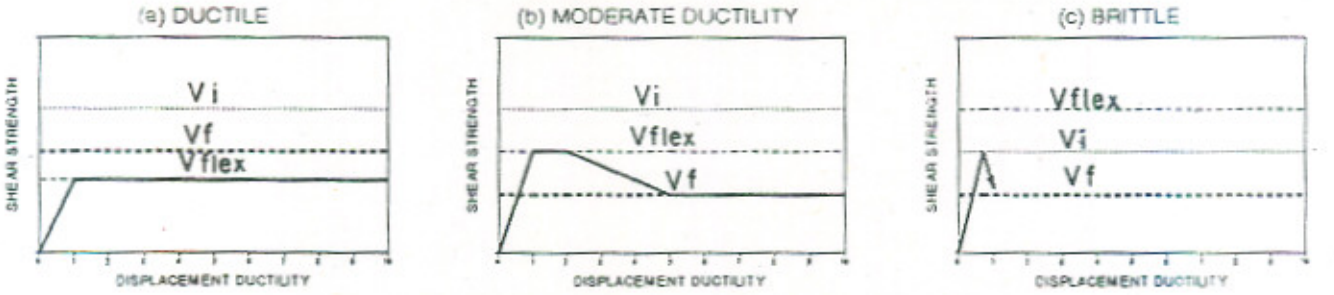



Fig. 9: Classification of Failure Behavior

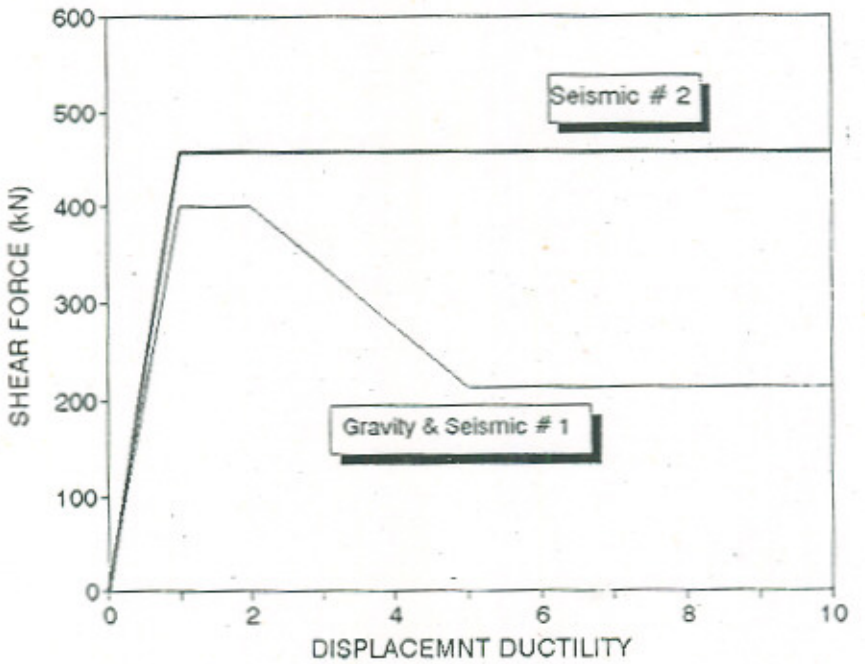



Fig. 10: Comparison of Failure Behavior for the 3 Designs