Shake-Table Testing of U.S. and Japanese Bridge Column Design

by
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ABSTRACT

This paper described a preliminary finding of shake-table tests of three one-sixth scale models that were designed from a U.S. and Japanese cooperative study in comparative seismic design of bridge columns. This study was undertaken by the Federal Highway Administration's (FHWA) Turner-Fairbank Highway Research Center and Japan's Public Works Research Institute (PWRI). Test results reflected the different design approaches, and earthquake patterns (strong ground motion data) occurred in this two countries. Future investigations needed of the test results were discussed.

KEYWORDS: Seismic Design; Comparative Design; Shake Table Test; Bridge Design; Residual Displacement.

1. INTRODUCTION

1.1 Background

Recently, three large destructive earthquakes have occurred in the United States and Japan. These earthquakes, the Loma Prieta earthquake in 1989, the Northridge earthquake in 1994, and the Kobe earthquake in 1995, have severely damaged a number of highway bridges, and have cost many lives and billions of dollars. Although, most of the damaged bridges were designed and constructed prior to the implementation of modern seismic design codes, significant damage also occurred in some bridges built to more recent codes. Column failure due to insufficient ductility and inadequate shear reinforcement was found to be the most important factor causing bridge collapse or failure.

In order to better evaluate the current bridge seismic codes, and to improve bridge performance under earthquakes, a cooperative research study of the highway bridge columns was undertaken by the Federal Highway Administration's (FHWA) Turner-Fairbank Highway Research Center and Japan's Public Works Research Institute (PWRI). Under this cooperative research project, a review of seismic design philosophy and criteria for bridges of both countries was conducted, and a comparative design study for a representative bridge column was performed using the present bridge design

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specifications (at that time). [Note: Japan has revised the seismic design specifications for bridges due to the Kobe earthquake, and issued in November 1996.]

Seismic design specifications used in this comparative study are: the newly issued (Spring 1995) Division I-A: Seismic Design of the AASHTO Standard Specifications for Highway Bridges and Part V: Seismic Design of the Japan Road Association's (JRA) Specification for Highway Bridges, issued in 1990. As a follow up to this comparative design, three one-sixth scale models were constructed and tested on a shake-table. The preliminary findings of the shake-table test of these three scaled models are presented in this paper. The details of the prototype bridge columns developed by the comparative designs, such as column sizes, main and transverse reinforcement, and the design approaches were compared. Further steps to investigate the results (data) of the shake-table test were discussed.

1.2 Objectives

The objectives of this cooperative research study are (1) to better assess the current seismic design codes in column design, and (2) to improve bridge performance under earthquakes. The shaking-table test results of the scale models will be evaluated to give an indication of the adequacy of current design criteria for bridge piers. .

2. DESIGN COMPARISON

For obtaining an actual comparison of these design codes, a bridge representative of the most common type in both countries was selected and shown in Figure 1. A comparative design based on the same force level were designed. The detailed comparison of the bridge columns seismic designs were compared by Yen et al (1996). The fundament characteristics, such as sizes, reinforced steels were shown in the Table 1. Following by this prototype design comparison, three scaled models were constructed according to these prototype designs.

As part of the comparison of results and exchange of design examples, a parametric study of different sizes was performed by both sides to make the comparison between the two design codes more meaningful. However, during the designing process, a temporary guideline of bridge seismic designs were issued after the Kobe earthquake. This temporary guideline was then included into this competitive design to check two different natural period type designs. In Table 1 the AASHTO bridge column size was 7 ft (2.1 m) in diameter, while the JRA columns were 2.7m (9ft), and 3.2 m (10.5 ft) in diameter.

3 SHAKE-TABLE TESTS

3.1 Test Setup

All scaled models were tested in the shake-table at the Vibration Laboratory of the Public Works Research Institute. These tests were to examine how the bridge columns behaved under a real earthquake-type shaking. Test setup is shown in Figure 2. The bottom of testing specimen (scaled column) was anchored at the center of the shake-table, and the top of the specimen was connected by two simply supported girders. The total length of the model bridge is 17m, and a total dead load weighted 26.8tf of the superstructure was placed on the girders. The concrete column specimen and girders were connected by pin supports and the other ends of girders were supported by low friction roller bearings on the steel frames. These steel frames were away from the shake table to allow the girders to move in the longitudinal direction. All excitations were given along the longitudinal direction axis of the model bridge. All the inertia forces during the excitation were considered to be carried by the model column. A total of 86 channels of electrically monitoring instrumentation were installed on each test specimen. These included 60 strain gages on the longitudinal and transverse reinforcement, five accelerometers to measure acceleration, nine displacement transducers were used to measure the absolute and relative horizontal displacement, and twelve transducers were used to determine the curvature of the column at six levels.

3.2 Test Specimens Design

Considering the available facilities, the scale factor of model design was determined to one-sixth of the prototype. The height of the column (from the supporting pin to the bottom of the column) is then defined as 2,442mm which is obtained from original height 14,650mm divided by this scale factor. The diameter of each column is then determined as 356mm, 450mm and 533mm, respectively. Dimension analysis was designed to determined the input acceleration so that the scale down of time-axis and the mass of superstructure could be determined. The scale factor of acceleration is 1, but the time axes need to be reduced to $1/\sqrt{6}$ and the mass of superstructure is obtained as 26.8tf. This value is an approximate value of the prototype weight 960tf dividing by $(6)^2$. To simulate the actual conditions as closely as possible, the specimens were designed such that the model columns have the same longitudinal reinforcement ratio and the same volumetric ratio as the prototype columns. The model specimens are given in Table 2.

3.3 Shaking Table Test Program

A series of excitations was applied to test each specimen under different level's ground motions. The revised Kaihoku-bashi record was used as excitation acceleration record. This is because that this record was specified as a standard ground motion for the type-I soil in the Design Specifications for Highway Bridges, Part V- Seismic Design. The maximum response acceleration spectrum with a damping ratio of 0.05 of the specified motion was 0.2g. The time scale of acceleration was compressed to $1/\sqrt{6}$, so that the acceleration of the shake table could simulate as closely as possible to the original conditions.

Each excitation increased in intensity of maximum acceleration response spectrum (ARS) from 0.2g to 1.0g by 0.2g increment. The maximum 1.0g excitation level was determined by the stroke limit of the shake table facility. At the maximum ARS, five excitations were tested. Since tests under the design load, each of specimens did not reach to the ultimate stage after the test sequence indicated, an additional mass was added to the superstructure (from 26.8tf to 40.0tf). With carrying a mass of 40.0tf, another sequence of ground motion was applied from a single cycle of the maximum ARS of 0.2g to five cycles of the maximum ARS of 1.0g. In the case that the specimen did not reach the ultimate stage after the five cycles of excitation, the excitation would be continuing until the specimen reach the ultimate stage or the maximum excitations of twenty. Ultimate stage was defined as a column specimen loses major bearing capacity.

3.4 Test Results

U.S. Specimen, (D=356mm)

First flexural cracking of concrete was occurred at the bottom of the column when the maximum ARS of 0.4g was applied. Flexural cracks extended up to an approximate height of 0.6m after the fifth excitation of ARS of 1.0g was applied. After adding the mass to 40tf, the spalling-off of cover concrete was recorded when the fifth excitation of ARS of 1.0g was applied. No longitudinal reinforcement fractured at the time of the eighteenth excitation of ARS of 1.0g was applied. The test was completed at this point because of the displacement limitation of the roller supporting was reached.

Japanese Specimen (1), (D=450mm)

First flexural concrete cracking was observed at the bottom of the column when the maximum ARS of 0.4g was applied. Flexural cracks extended up to an approximate height of 0.5m after the fifth excitation of ARS of 1.0g was applied. After adding the mass to 40tf, the spalling-off of cover concrete was recorded when the second excitation of ARS of 1.0g was applied. One of longitudinal reinforcement fractured when the seventh excitation of ARS of 1.0g was applied.

Japanese Specimen (2), (D=533mm)

First flexural cracking was noticed at the bottom of the column when the maximum ARS of 0.2g was applied. Flexural cracks extended up to an approximate height of 0.6m after the fifth excitation of ARS of 1.0g was applied. After adding the mass to 40tf, the spalling-off of cover concrete was recorded when the fourteenth excitation of ARS of 1.0g was applied. One of longitudinal reinforcement fractured when the eighteenth excitation of ARS of 1.0g was applied.

3.5 Load-Deformation Characteristics

The horizontal load-deformation hysterisis at the top of three columns are shown in Figure 3-5 respectively. Each figure implies the load-deformation hysterisis of the maximum ARS of 0.4g and the first and fifth excitation of the maximum ARS of 1.0g. The horizontal loads are calculated by multiplying the measured acceleration by the mass. In all cases, the hysterisis loop of the maximum ARS of 0.4g is almost elastic, while the loops of the first and fifth excitation of the maximum ARS of 1.0g show the behavior of the good energy absorption. Figure 3 shows that the fifth excitation of 1.0g have the residual displacement while the first excitations of 1.0g have almost no residual displacement for the U.S. specimen. Figure 4 and 5 show that the two Japanese specimens have almost no residual displacement.

3.6 Residual Displacement

The recorded residual displacements after each excitation are shown in Figure 6. Three specimens have almost the same residual displacement up to the 11^{th} excitation, which was the first excitation after adding the mass. The residual displacement of U.S. specimen increased largely after the 13^{th} excitation, compared with the two Japanese specimens. However, the U.S. specimen maintained the adequate capacity until the completion of the test while the residual displacement reached 120mm (which is almost 6% of a column height). On the other hand, Japanese specimen (1) could not provide the sufficient capacity at the residual displacement of 40mm which is 1.6% of a column height. The residual displacement of Japanese specimen(2) also gradually increased and reached the ultimate stage at the residual displacement of 60mm that is 2.5% of a column height. The U.S. specimen has a larger residual displacement, but did not reach the ultimate stage until the last excitation.

4. CONCLUSIONS AND RECOMMENDATIONS

This paper presented design comparisons of design principles and results from seismic design of a typical bridge column by both US and Japanese methods. The results here do not seek to show the stronger or weaker design but rather to compare the different design approaches, and to learn from each. From the natural periods compared in Table 1, the AASHTO pier is more flexible than the JRA piers in this case. These designs may also reflect the difference in the strong ground motion, which is affected most by the soil profile type.

From the results of shaking-table test, three 1/6 scale column models performed very well under the design loads. There was no critical damage, such as fracture of longitudinal rebars, in the design level test. After increased to approximate 150% of the design loads, results show US specimen has no critical damage till the limitation of the shake-table's displacement was reached. However, the residual displacement increased more than the other two Japanese specimens. The Japanese specimens experienced fracture of longitudinal rebars at the bottom connections. Major difference in the design detail is the transverse confinement. The U.S. uses spiral type with closer spacing instead of hoop type with relatively larger spacing from Japanese design, and may provide sufficient ductility under large horizontal displacement. Compared with a simple static and dynamic analytical model, shake-table test results agree each other very well. Much more detailed analyses are needed to investigate the differences of dynamic behavior among three specimens.

During the shake-table tests, 86 channels' data sets were recorded for each specimen. These data included curvature displacement of model columns, contain much information of bridge column's performance in a simulated earthquake event, particularly, while bridge column receives much greater earthquake load than design load. Nonlinear finite element model or mathematical modeling would be very helpful to examine the bridge model dynamic characteristic. Thus two analytical modeling are recommended to be used in examining the data.

For Finite Element Modeling, a

nonlinear program such as SAP2000 is expected to model the bridge column in detail dynamic response. In the System Identification Method, a generalized multiple input and multiple out system identification method developed for bridge structures (Yen, et al, 1996) will be used to assess the dynamic parameter of the model bridge. This system identification method uses both excitation forces (input data) and structural dynamic response (output) to estimate the system transfer function.

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Table 1. Comparison of Design Results

	U.S. (AASHTO) DESIGN	JAPANESE DESIGN (1)	JAPANESE DESIGN (2)
Column Size	Diameter = 7 ft (2.1m)	Diameter = 2.7 m (9 ft)	Diameter = 3.2m (10.5ft)
Main Reinforcement (longitudinal direction)	56-#18 bars (56-D57)	2X54-D51 bars (2X54-#16 Double layer)	2X32 -D32 bars (Double Layer)
Transverse Reinforcement	#7 @ 3"(D22@75) (Spiral type)	D22 @150 (#7@6") (Hoop type)	D19 @150 (#7@6") (Hoop type)
Fundamental Natural Period (longitudinal direction)	1.32 sec	0.93 sec	0.85 sec
Axial Load	698tf	795tf	894tf
Concrete Gross Area	38.48 ft^2 ($3.57m^2$)	$5.73m^2$ (61.63 ft^2)	8.04 m^2

Table 2. One-Sixth Model Columns

	U.S. DESIGN	JPAANESE DESIGN (1)	JAPANESE DESIGN (2)
Diameter of Column	D=356mm A=995 cm^2	D=450 A=1590cm^2	D=533 A=2231cm^2
Longitudinal Reinf.	D19 X 14 As= 40.10 cm^2 (ρ=4.03%)	D13 X 24X2 As= 60.82cm^2 (ρ=3.80%)	D10X25X2 As= 35.61cm^2 (ρ=1.60 %)
Transverse Reinf.	D6@40 (spiral) ρ_s = 1.02%	D6@85 (double hoop) ρ_s = 0.84%	D6@110 (double hoop) ρ_s = 0.52%
Axial Load and Stress at Bottom (Dead Load)	Pdead = 14.25tf σdead = 14.3kgf/cm^2	Pdead = 14.46tf σdead = 9.1kgf/cm^2	Pdead = 15.06tf σdead = 6.7kgf/cm^2

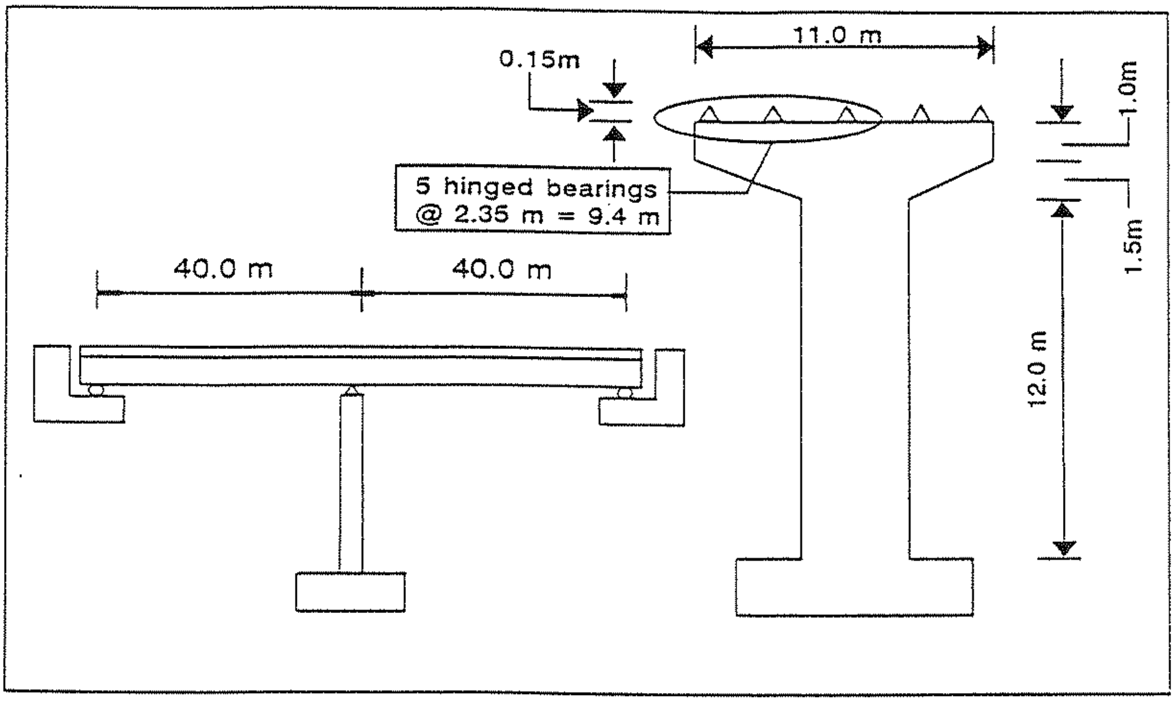

Figure 1. The Selected Bridge Column Design Dimensions

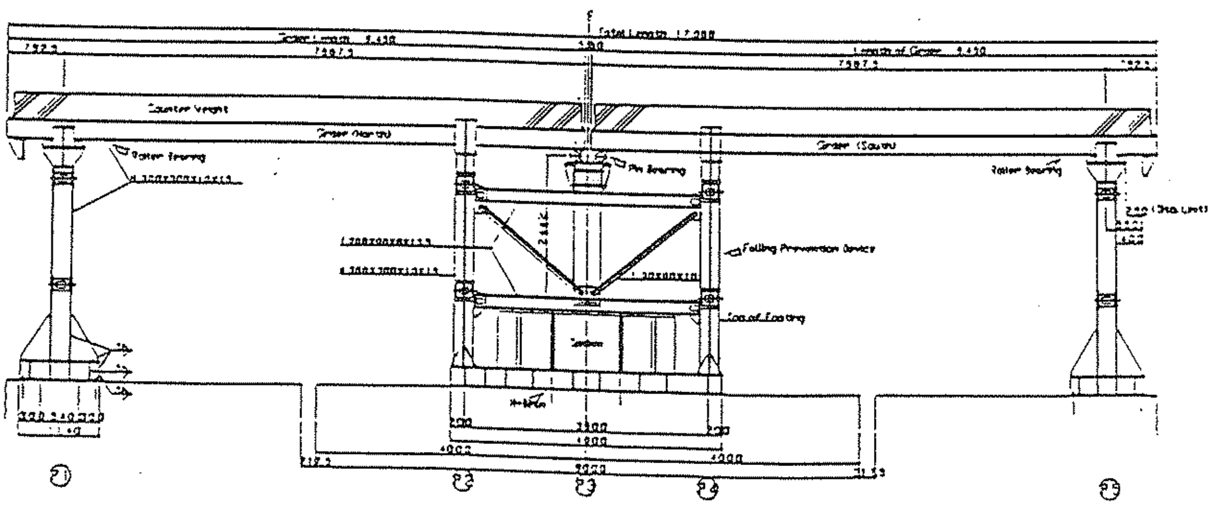

Fig 2 Test Setup

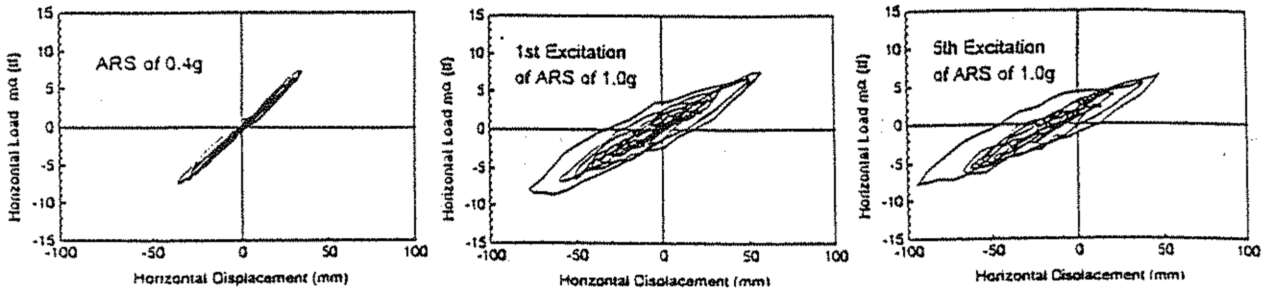

Fig.3 Load -Displacement Hysteresis (ϕ356)

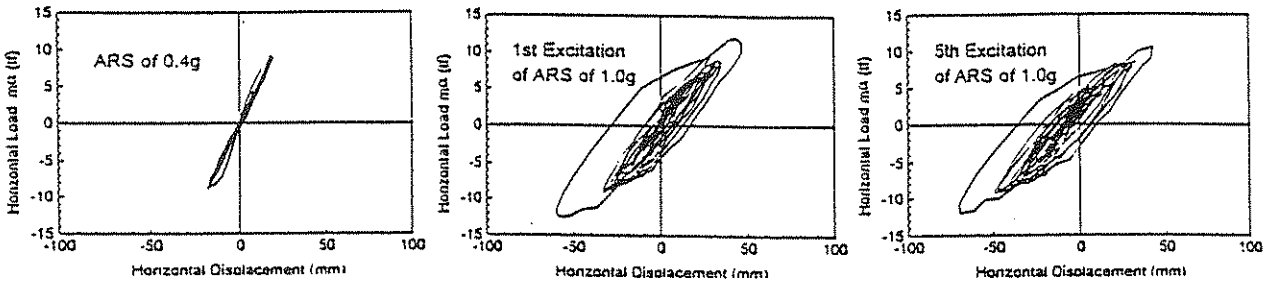

Fig. 4 Load -Displacement Hysteresis (ϕ450)

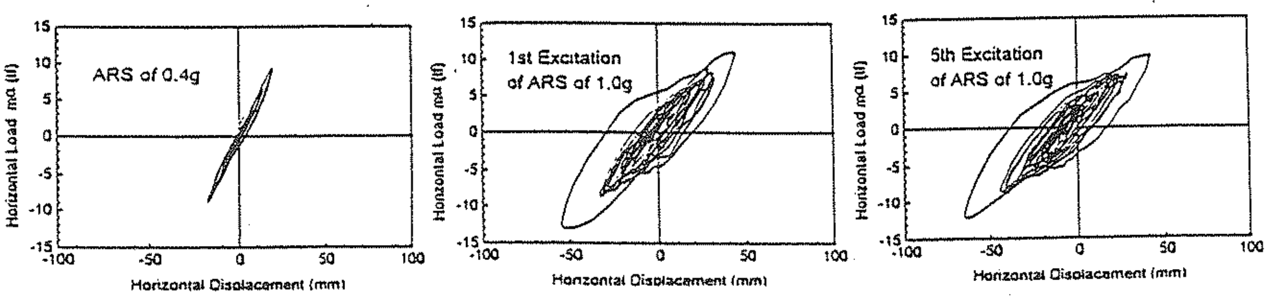

Fig. 5 Load -Displacement Hysteresis (ϕ533)

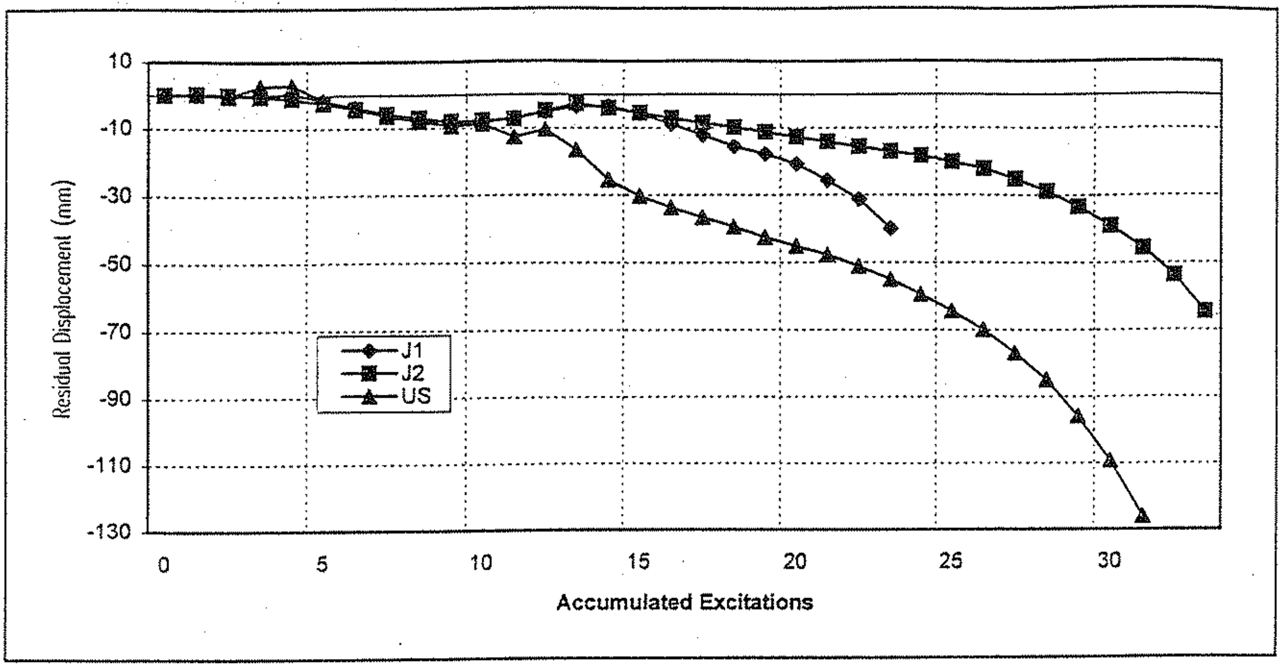

Figure 6 Residual Displacements