Assessment Performance of Ductile and Damaged Protected Bridge piers Subjected to Bi-Directional Earthquake Attack

Manas Hardaha[1] Shikha Shrivastava[2]
[1]Research Scholar [2]Assistant Professor
[1,2]Department of Civil Engineering
[1,2]Saraswati Institute of Engineering & Technology, Jabalpur, India

Abstract— Incremental Dynamic Analysis (IDA) procedures are advanced and then applied to a quantitative risk assessment for bridge structures. This is achieved by combining IDA with site-dependent hazard-recurrence relations and damage outcomes. The IDA procedure is also developed as a way to select a critical earthquake motion record for a one-off destructive experiment. Three prototype bridge substructures are designed according to the loading and detailing requirements of New Zealand, Japan and Caltrans codes. From these designs 30 percent reduced scale specimens are constructed as part of an experimental investigation. The Pseudo dynamic test is then to control on three specimens using the identified critical earthquake records. The results are presented in a probabilistic risk based format. The differences in the seismic performance of the three different countries' design codes are examined. Seismic response is expected to be resulting damage on structures, which may threaten post-earthquake serviceability. To overcome this major performance shortcoming, the seismic behaviour under bi-directional lateral loading is investigated for a bridge pier designed and constructed in accordance with Damage Avoidance principles. Due to the presence of steel armoured rocking interface at the base, it is demonstrated that damage can be avoided, but due to the lack of hysteresis it is necessary to add some supplemental damping. Experimental results of the armoured rocking pier under bi-directional loading are compared with a companion ductile design specimen. The Pseudo dynamic (PD) test method was developed about 30 years ago by Takanashi et al. (1975) and is thought to be the most efficient and powerful alternative to both STT method and dynamic analytical method, especially when the real response behaviors, such as the damage state during and after a certain earthquake are need to be investigated. On the other hand, considerable number of the dynamic analysis programs running on conventional personal computers has been developed recently and the accuracy and reliability of the results improved as the new theories are applied to them. Also, the cost of running the computer becomes cheaper. Considering these background, the dynamic analysis is strong and reasonable tool for the seismic research except that the dynamic analysis method is needed to assume the simplified model for the properties of structures such as the lateral load and displacement relationship and hysteresis damping factors. On the other hand, considerable number of the dynamic analysis programs running on conventional personal computers has been developed recently and the accuracy and reliability of the results improved as the new theories are applied to them. Also, the cost of running the computer becomes cheaper. Considering these Back ground, the dynamic analysis is strong and reasonable tool for the seismic research except that the dynamic analysis method is needed to assume the simplified model for the properties of structures such as the lateral load and displacement relationship and hysteresis damping factors. As mentioned previously, this research explores the use of a newly developed Seismic Risk Assessment (SRA) methodology. This proposed methodology can be applied Performance-Based Earthquake Engineering as a tool to estimate the damage outcome corresponding to a certain level of an earthquake. Furthermore, this methodology makes it possible to select a critical input earthquake motion for a one off Experiment.

Key words: Incremental Dynamic Loading, Pseudo Dynamics, Seismic Response, Risk Assessement, Serviceability, Bi-Directional Loading, Hazard-Recurrence

I. INTRODUCTION

Incremental Dynamic Analysis (IDA) is applied in a Performance-Based Earthquake Engineering context to identify critical earthquake ground motions that are subsequently to be used in physical testing or analytical studies to investigate structural response and damage outcomes. This quantitative risk analysis procedure consists of choosing a suitable suite of ground motions and appropriate intensity measures; performing IDA on a nonlinear model of the prototype structure; summarizing the IDA results and parameterizing them into 10th, 50th, and 90th percentile performance bounds; integrating these results with respect to hazard intensity recurrence relations; and identifying the strength of two or three critical earthquakes that will potentially encompass all damage states through to collapse. An illustrative example of the procedure is given for reinforced concrete highway bridge piers, designed to New Zealand, Japan and Caltrans specifications.

Performance Based Earthquake Engineering (PBEE) procedures require the prediction of the seismic capacity of structures which is then compared to the local seismic demand. The interrelationship between the two gives an inference of the expected level of damage for a given level of ground shaking. Incremental Dynamic Analysis (IDA) is a new methodology which can give a clear indication of the relationship between the seismic capacity and the demand. Engineers can estimate principal response quantities, such as the maximum drift of the structure for a given intensity measure (IM) such as peak ground or spectral acceleration. The need to identify a critical earthquake for the purpose of an experimental investigation or further advanced analysis and design can be accommodated by the application of IDA. A synthesis of IDA curves into 10th, 50th, and 90th percentile bounds helps the designer to single out critical ground motions which can then be used in physical testing or advanced analysis to investigate structural damage with a certain level of confidence.

In order to investigate the likely seismic performance of multi-storey precast concrete buildings, Matthews (2004) adopted the method given by Equation. Then based on analysis he derived a protocol for super-assemblage specimen testing that was a physical representation of a family of typical prototype precast concrete buildings. In order to estimate structural performance under seismic loads, Vamvatsikos and Cornell (2004) presented a procedure called "Incremental Dynamic Analysis (IDA)". This approach involves performing nonlinear dynamic analyses of a prototype structural system under a suite of ground motion records, each scaled to several intensity levels designed to force the structure.

II. LITERATURE REVIEW

A. KEVIN SOLBERG, N.MASHIKO

Recent earthquakes such as Loma Prieta, Northridge, and Kobe have demonstrated a need for a new design philosophy of bridge piers that avoids damage in order to ensure post-earthquake serviceability and reduce financial loss. Damage Avoidance Design (DAD) is one such emerging philosophy that meets these objectives. DAD details require armoring of the joints; this eliminates the formation of plastic hinges. Seismic input energy is dissipated by rocking coupled with supplemental energy dissipation devices. In this paper the theoretical performance of a DAD bridge pier is validated through bi-directional quasi-static and pseudodynamic tests performed on a 30% scale specimen. The DAD pier is designed to rock on steel-steel armored interfaces.

B. Shriharsh Satish Modak

Bridges are very important structures and play important role during an earthquake for evacuation of people as well as in the post-earthquake events. Pile and well foundations are the two types of deep foundation, mostly used for both the railway and road bridges spanning the river. Due to the availability of manpower and skill for construction, the well foundation has been more popular in India. Since the well foundations are massive structures, deeply embedded in soil, it was believed that they are immune to seismic damage.

C. Shijiazhuang

Plastic hinge model has been widely used in bridge seismic design codes such as Japan, Caltrans, New Zealand and China (revised edition), to evaluate deformation capacity of RC bridge columns. With the development of bridge performance/displacement based seismic design, several damage indices have been suggested, such as ultimate curvature and curvature ductility factor of critical section, maximum strain of confined concrete and reinforced steels, low cycle fatigue damage indices of longitudinal reinforcement etc.

D. Diwaker Katiyar

Reinforced concrete (RC) frame members under seismic loads are likely to experience large inelastic deformations and therefore, adequate ductility is essential to avoid brittle failure mode and enhance energy dissipation potential. The satisfactory post-yield performance of these RC members during a seismic event largely depends on the characteristics of material used in their fabrication, namely, reinforcing steel and concrete. This study is concerned with the effect of reinforcing steel characteristics and their manufacturing process on the flexural behavior of beams up to failure. Properties of reinforcing steel bars which could affect the seismic behavior of structural members are yield strength (YS), ultimate strength (UTS), strain value at which strain-hardening commences, UTS/YS ratio, fracture strain and their dependence on the manufacturing process, such as cold-twisting deformed(CTD) vs. quenched self-tempered (QST) or thermo-mechanically treated (TMT). Effect of reinforcing steel and concrete properties on ductility and moment resisting capacity was studied using moment-curvature analyses of RC beam sections which considered nonlinear behavior of concrete and actual stress strain curve of individual reinforcing steel bars.

III. SCOPE OF PRESENT WORK

This paper introduces a seismic hazard examination philosophy. The reason for this advancement is to choose ground movements that are unimportant for trial examinations. This is turned out to be in ruinous model tests there is just a single chance to accurately recognize execution modes and general conduct properties. Consequently it is basic that such a test be attempted once more into a hazard base setting where a dimension of certainty can be communicated in the result. The paper exhibits a test examination where the seismic execution of three scaffold docks intended to New Zealand, Japan and Caltrans details. Harm states following DBE and MCE are explored. Part 4 shows an elective perspectives on how connect is utilized to create and developed. Harm Avoidance Design is utilized to build up an exploratory example. The base of the extension wharf is "Harm Protected" by utilizing reinforced subtleties alongside vitality dissipators.

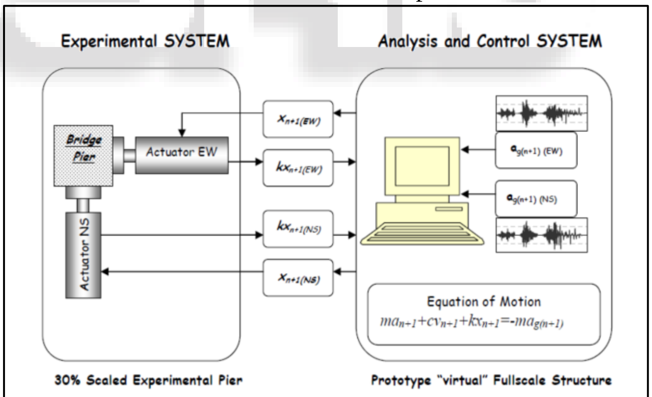
IV. METHODS & MATERIALS USED

A. Quantitative Risk Assessment

1) Stage 1: Select ground movement records and demonstrating the structure In request to perform IDA, a suite of ground movement records are required. A similar ground movements were embraced for this investigation. These tremors have Richter sizes in the scope of 6.5-6.9 with moderate epicentral separates generally in the scope of 16 to 32 km; all these ground movements were recorded on firm soil, once in a while alluded to as the scattering, over the range. For this suite of seismic tremors it is clear that the PGA fills in as a proper IM, gave that the period is not exactly about 1.6 seconds. A nonlinear computational model of the model basic framework should then be created. A check ought to be made that the scattering of reaction request (βD) in the area of the normal time frame is sensible. In the event that the scattering is over the top, at that point an option IM ought to be considered and this progression ought to be rehashed until the scattering is sensible.
2) Stage 2: Perform Incremental Dynamic Analysis Once the model and the ground development records have been picked, IDA is performed. To start the examination, the seismic tremor record picked must be scaled from a low IM to a couple of higher IM levels until assistant

breakdown occurs. For each expansion of IM, a nonlinear ground-breaking time history examination is performed. Examinations are reiterated for higher IM's until helper breakdown occurs. Finding the most extraordinary buoy found in an examination gives one point in the PGA versus most noteworthy buoy plot space. Results usually show a lognormal course of buoy (dislodging) results.

3) Stage 3: Model the IDA bend and factual results In their past examination, Vamvatsikos and Cornell (2004) demonstrated their IDA bends by utilizing various introduction spline capacities. It is viewed as that such an estimate is bulky and not especially valuable for resulting examination.. On the off chance that the info IM is more noteworthy than the "basic" esteem (for example Sa >Sc) at that point the reaction is with the end goal that θ > 2θc and auxiliary unsteadiness (breakdown) is up and coming. In Equation (2-2) the three control parameters (Sc, r, and either θc or K) are evaluated utilizing nonlinear least squares investigation for every individual seismic tremor ground 2-7 movement IDA informational collection. Figure 2-3 (b) outlines the fit between the IDA information focuses and the fitted R-O bend for one explicit case. Despite the fact that the outcomes for every one of the control parameters are unique, they would then be able to be inspected all things considered and a measurable examination on the parameters can be performed. Studies demonstrate that the parameters are log typically appropriated. Along these lines by finding out middle estimations of every parameter the 50th percentile IDA reaction can be spoken to by an individual R-O middle bend. In like manner by looking at inconstancy of individual IDA conveyances, parameters that speak to bends of different limits of intrigue, for example, the tenth and 90th percentiles might be found.
4) Stage 4: Assign harm limit states Once the three (tenth, 50th and 90th percentile) lines have been produced, it is conceivable to decide the normal float for a quake with a specific dimension of the force. Developing global best practice for seismic plan is having a tendency to receive a double dimension force approach that is (I) a DBE spoken to by a 10% in 50 years ground movement; and (ii) a MCE spoken to by a 2% in 50 years quake. A few harm limit-states can be characterized on the IDA bends created. In their past research, Vamvatsikos and Cornell (2004) connected structure use criteria of Immediate Occupancy (IO) and Collapse Prevention (CP) limit-states to their IDA bends dependent on structure use criteria. In this investigation, the meanings of harm limit states were reached out by receiving Mander and Basoz (1999) meanings of harm states for scaffolds, as recorded in Table 2-3. Two harm states can be effectively characterized as pursues. 2-8 DS=1 is for flexible conduct, it in this manner finishes up at the beginning of harm which is best characterized at the yield float (uprooting) of the structure. Additionally, DS=5 starts at the beginning of breakdown, and as portrayed over this is best characterized when θ > 2θc. The other harm stages (DS=2, 3, and 4) are increasingly emotional in their definition. It is recommended that the limit isolating DS=3 and DS=4 be characterized at that dimension of float where the structure would be regarded to have endured hopeless harm with the end goal that the structure would probably be relinquished; as prove by: (I) inordinate lasting float toward the finish of the seismic tremor; (ii) extreme harm to basic components, for example, clasping of longitudinal strengthening bars or the break of transverse circles as well as longitudinal fortifying bars. At last, the limit isolating DS=2 and DS=3 ought to be characterized as that dimension of harm that would require impermanent loss of capacity because of fixes that should be attempted for strengthened solid structures, this typically happens when spalling of spread cement is apparent. This relocation can likewise be found by examination when the spread solid pressure strain surpasses the spalling endure say εspall=0.008 at floats beneath this limit (i.e., DS=2) harm is viewed as slight and bearable. The consequence of relegating harm states to the IDA fractile bends.
5) Stage 5: Hazard– Recurrence Risk Relation Moreover, the computational demonstrating, in spite of the fact that it might be refined, isn't correct; there is a proportion of vulnerability that exists between the anticipated and the watched reaction. To envelop the irregularity of seismic interest alongside the inborn haphazardness of the auxiliary limit and the vulnerability because of estimation of the computational demonstrating it is important to utilize a coordinated methodology as recommended by Kennedy et al (1980). For nitty gritty appraisal, extra certainty interims can likewise be plotted with the 95th, 80th, 70th and 60th percentile bends.



(b) Bi-directional Pseudodynamic Test Concept at the $(n+1)^{th}$ step

Fig. 1: Psuedo-dynamic Test

V. CALCULATION OF PERFORMANCE OF PIERS

It present the seismic execution of the three scaffold wharfs (SP-1, 2 and 3), intended to the New Zealand, Japan and Caltrans codes. Every one of these figures of results presents:
1) an arrangement perspective on the bidirectional circle of float reaction,
2) load-relocation hysteresis bends,
3) the time-history of reaction float as per the three tremors,
4) Show photos of specific harm states amid every tremor (as characterized by the HAZUS (1999).

All through this segment, just the EW reaction of the extension wharfs is talked about in detail since the basic quake part was adjusted to the EW course. The harm states examined were yielding of bars, breaking, spalling of spread cement, clasping of bars, and cracking of bars. Yielding was made a decision after the tests from the information estimated by the vertical potentiometers. All other harm states were overviewed by visual perception amid the test. Moreover, the most extreme float, the greatest parallel burden amid every quake and the leftover float after every seismic tremor are likewise depicted in this segment.

A. New Zealand Bridge Pier

Figure exhibits the test consequences of the seismic execution of the New Zealand connect dock. Note that part (d) and (e) of Figure show photos of bar clasping at a float of 3.6% and toward the finish of the test indicating low cycle weariness cracks of longitudinal bar. The seismic execution of the New Zealand connect wharf is portrayed in detail dependent on harm saw after every quake as pursues: 3-9

EQ1 (0-25sec) When made a decision from the longitudinal bar strain surmised by outside instrumentation, yield happened at 5.61sec when the float surpassed 0.30% eastwards. The parallel burden when the wharf yielded was 63.3kN. A few level splits were watched 150mm separated amid the test, yet these breaks shut after the tremor stopped. The most extreme float and horizontal burden estimated were - 1.65% at 13.83 sec and 159 kN at 6.2 sec, separately. The remaining float was - 0.167%. The harm state after EQ1 (PGA=0.4g for the 90 percentile DBE) was surveyed as slight, that is DS2, since the wharf surpassed the yield float esteem and breaks showed up.

EQ2 (25-50sec) The most extreme float (- 2.48%) happened at time=36.9sec with level flexure splits dispersed around 50 mm separated over the lower 2D territory (roughly 1m) of the dock. The breaks were observed to be more serious than for EQ1 however the remaining split width was still generally little (not more than 0.2 mm). The spread cement stayed in tasteful condition and no spalling was watched. The remaining float was - 0.25%. The harm state following EQ2 (PGA=0.8g for the 50 percentile MCE) stayed at DS2.

EQ3 (50-100sec) The significant harm occasions saw under EQ3 were spalling, clasping, starting bar cracks and extreme bar breaks, which brought about the quality debasing quickly compelling the end of the test. The first spalling and bar clasping happened on the East essence of the wharf at 63.7 sec with 2.5% float and 68.4 sec with 3.6% float, separately. Consequently, the primary bar crack happened at 71.7sec at a float of 6.0%. This was effectively distinguished by a slamming commotion together with an unexpected drop in horizontal burden opposition. The real corruption of solidarity began at 74.5 sec when the highest point of the dock was at a float of 6.52%. From there on, the horizontal burden quality of the wharf diminished to 80% (from 78.7kN to 62.6kN), while the float of the dock expanded 1.75% (from 6.53% to 8.27%). This corruption wonder was surveyed as a 3-10 genuine harm flagging a potential breakdown of the dock and the test was ended. Because of a compelling breakdown; it was unmistakably clear that the harm state was DS=5.

B. Japanese Bridge Pier

Figure shows the trial consequences of the seismic execution of the Japanese scaffold wharf. The two photos appear (d) spread spalling at a float of 2.7%; and (e) the example toward the finish of testing. Subtleties of the seismic execution of the Japanese scaffold dock appearing of the three quake pursue:

EQ1 (0-25sec) The Japanese extension dock (SP-2) yielded when the float came to - 0.20% at 5.61 sec. Amid the tremor, two main flat breaks framed, one at the base of the wharf and the other 300 mm from the base; anyway these splits shut after the seismic tremor. The most extreme float and the relating parallel burden estimated were - 1.48% and - 327 kN at 13.02 sec separately. The lingering float was - 0.05% demonstrating slight harm, in this manner DS=2 for the DBE.

EQ2 (25-50sec) The greatest float (- 1.76%) was estimated when the parallel burden was - 355kN at 30.18sec with even breaks roughly 100 mm separated showing up all through the bottommost 60 cm (equivalent to the distance across of the dock). The splits were observed to be more concentrated than those under EQ1. By and by, no leftover breaks were unmistakable. The leftover float was - 0.11% and the harm state after EQ2 stayed at DS=2.

EQ3 (50-100sec) The degree of harm coming about structure EQ3 was confined to cover concrete spalling, which happened at 66.1 sec at a float of 2.7%. Toward the finish of the tremor, the remaining float 3-11 was 0.11%. As certain fixes were important to reestablish full working request the harm state are estimated as DS=3.

C. Caltrans Bridge dock

Figure demonstrates the trial consequences of the seismic execution of the Caltrans connect dock. A visual perspective on the degree of harm in the later piece of the investigation. The seismic execution of the Caltrans connect dock for every quake is depicted as pursues:

EQ1 (0-25sec) The Caltran's wharf (SP-3) yielded at 5.58sec with - 0.24% float when the horizontal burden was - 93.9kN. A few even breaks divided each 200 mm from the base of the wharf were found amid EQ1, yet these splits shut after the seismic tremor ended. The most extreme float and the relating parallel burden were 1.53% and 232kN separately estimated at 13.11sec. The lingering float was - 0.12% demonstrating slight harm, in this manner the harm state was DS=2.

EQ2 (25-50sec) The most extreme float (1.95%) was estimated at 36.0 sec with even breaks (roughly 50 mm separated) around the bottommost one-distance across range (0.6 m) of the wharf. The sidelong burden comparing to the greatest float was 259 kN. The splits were observed to be more serious than those under EQ1 yet at the same time they were under 0.1mm in width. The spread cement stayed unblemished and no spalling was watched. The lingering float was - 0.13% with the harm state staying at DS=2.

EQ3 (50-100sec) The harm occasions found under EQ3 were spread cement spalling and bar clasping on the East side. The spalling and clasping happened at 65.1 sec and 70.2 sec when the wharf floats were 3.7% and - 5.29%, individually. The remaining float was - 0.20%. As the harm to 3-12 the longitudinal clasped bars lead to a hopeless condition, the harm state toward the finish of testing was viewed as DS=4. 3.6

D. Comparison of Each Pier's Seismic execution

Three examples under every one of the three progressive tremors are orchestrated independently to give nine power relocation hysteresis charts alongside three float time-history reactions. Under the DBE (EQ1) the heap uprooting connections demonstrate that all the extension docks displayed restricted hysteresis reaction and just a negligible leftover float stayed toward the finish of Earthquake. In spite of the fact that the firmness of the New Zealand dock (SP-1) is not exactly the others because of its littler measurement and lower parallel quality, the most extreme uprooting reaction of every wharf did not contrast significantly. The adjusted idea of the hysteresis circles close to their pinnacles is because of the marvel of "synchronous bi-directional cooperation impact" (Mutsuyoshi, 1994), where the sidelong burden in a specific course will in general be diminished by its symmetrical development. Examination of the float time-accounts, appeared in Figure .preceding the most extreme pinnacle reaction at roughly 13.7 sec, shows that the reactions of all wharfs were comparable, however because of various yield focuses after the pinnacle was attainted the reactions differed, especially the New Zealand dock (SP-1). Under EQ, it is fascinating to take note of that despite the past reaction, the reactions of the three wharfs are comparable from 25 to 37 sec until the principal expansive inelastic trip occurred. The reaction around then was biggest in the weakest and most adaptable of the three docks.

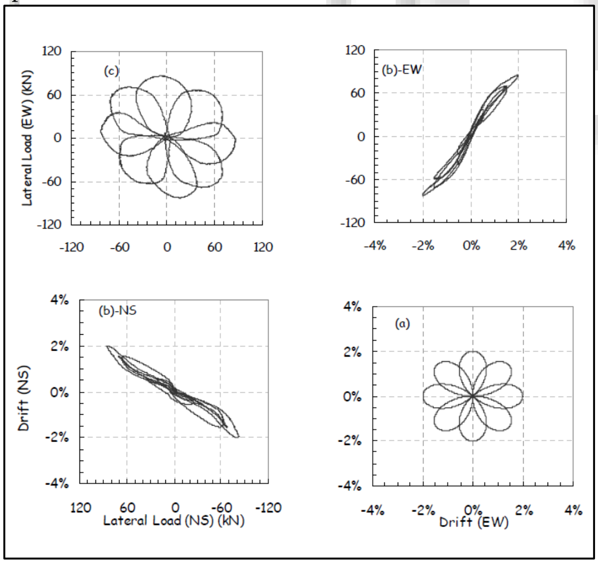


Fig. 2: Graphs/ Results Obtained

VI. CONCLUSIONS

In light of this hypothetical improvement and test examination revealed in this, the accompanying central ends are drawn:

1) Seismic Risk Assessment (SRA) approach was created by incorporating probabilistic repeat relations with and propelled IDA systems. In this manner dependent on given dimension of peril introduction it is conceivable to survey the harm result for a recommended level of certainty.
2) 2. Utilizing the created SRA approach, thinks about on the conduct of configuration to New Zealand, Japan and Caltrans determinations were completed both logically and tentatively. Results demonstrate that for every one of the three nations proprietor can clear up a high level of trust in the execution under DBE where just slight harm is normal alongside a fast come back to support. Be that as it may, under MCE occasions there is a sensible (30%) chance that hopeless harm or breakdown may happen.
3) 3. Harm Avoidance Design of extension wharfs offers a few favorable circumstances. As showed in the investigations the DAD dock performed will under both DBE and MCE, neither harm nor remaining float was recognized. The DAD dock had just 65 percent of the quality of the partner New Zealand planned extension wharf, however demonstrated that despite the fact that the floats were somewhere in the range of 15 percent more prominent there was no harm that would prompt loss of courtesy of the structure. Subsequently, as far as the prevalent post-seismic tremor functionality alongside the efficient advantages picked up from pre-throwing.

REFERENCES

[1] Chopra K.A., 2000, Dynamics of structures (second edition), Prentice Hall, New Jersey
[2] Housner, G.W., 1963, The Behaviour of Inverted Pendulum Structures During Earthquake, Bulletin of the Seismological Society of America, vol. 53, No. 3, pp. 403-417
[3] Mander J.B., Cheng C-T., 1997, Seismic Resistance of Bridge Piers Based on Damage Avoidance Design, National Centre for Earthquake Engineering Research, Technical Report NCEER-97-0014, December 10
[4] Skinner R.I., Robinson W.H., and McVerry G.H. 1993, An Introduction to Seismi Isolation, J. Wiley and Sons, England
[5] Takanashi K., Udagawa M., Seki M., Okada T., and Tanaka H., 1975, Nonlinear earthquake response analysis of structures by a computer-actuator on-line system, Bulletin of Earthquake Resistant Structure Research Centre 8 Institute of Industrial Science, University of Tokyo, Tokyo, Japan
[6] Tanabe, T., 1999, Comparative Performance of Seismic Design codes for Concrete Structures, Vol. 1, Elsevier, New York BIS. (2000). IS 456: Plain and reinforced concrete-Code of Practice, Bureau of Indian Standards, New Delhi, India.