



สำนักงานกองทุนสนับสนุนการวิจัย
THE THAILAND RESEARCH FUND

รายงานการวิจัยฉบับสมบูรณ์
โครงการวิจัยทุนส่งเสริมกลุ่มวิจัย (เมธีวิจัยอาวุโส สกว.)

โครงการ

การวิจัยและพัฒนาด้านการลดภัยพิบัติธรรมชาติ
เพื่อการพัฒนาที่ยั่งยืน

2540 - 2543

โดย

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มีนาคม 2545

Scan 

รายงานวิจัยฉบับสมบูรณ์

โครงการ การวิจัยและพัฒนาด้านการลดภัยพิบัติธรรมชาติ เพื่อการพัฒนาที่ยั่งยืน

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(ความเห็นในรายงานนี้เป็นของผู้วิจัย สกว. ไม่จำเป็นต้องเห็นด้วยเสมอไป)

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สัญญาเลขที่ RAT/03/2540

โครงการวิจัย : การวิจัยและพัฒนาด้านการลดภัยพิบัติธรรมชาติเพื่อการพัฒนาที่ยั่งยืน

Research and Development on Natural Disaster Reduction for Sustainable Development

สรุปผลงานของโครงการในรอบ 3 ปี 2540-2543

ชื่อหัวหน้าโครงการ ศาสตราจารย์ ดร.ปณิธาน ลักคุณะประสิทธิ์

รายงานในช่วงตั้งแต่วันที่ 1 ธันวาคม 2540 ถึงวันที่ 6 ธันวาคม 2544

1. ผลงาน

1.1 บทความที่ได้รับการตอบรับให้ตีพิมพ์ในวารสารนานาชาติ 5 บทความ (ภาคผนวก ก) ได้แก่

- ก. Lukkunaprasit, P., and Wanitkorkul, A. (2001) "Inelastic Buildings with Tuned Mass Dampers under Moderate Ground Motions from Distant Earthquakes" *International Journal on Earthquake Engineering and Structural Dynamics*, Vol. 30, 537-551.
- ข. Pinkaew, T., and Fujino, Y. (2001) "Effectiveness of Semi-Active Tuned Mass Dampers Under Harmonic Excitation" *Engineering Structures*, Vol. 23, 850-856.
- ค. Sittipunt, C., Wood, S.L., Lukkunaprasit, P., and Pattararattanakul, P. (2001) "Cyclic Behavior of Reinforced Concrete Shear Walls with Diagonal Web Reinforcement" *ACI Structural Journal*, Vol. 98(4), 554-562.
- ง. Teachavorasinskun, S., Thongchim, P. and Lukkunaprasit, P. (2001). "Shear Modulus and Damping Ratio of a Clay During Undrained Cyclic Loading" *Geotechnique*, Vol. 51(5), 467-470.
- จ. Teachavorasinskun, S., Thongchim, P. and Lukkunaprasit, P. (2002). "Stress rate effect on the stiffness of soft clay from cyclic, compression and extension triaxial tests" *Geotechnique*, Vol. 52(1), 51-54.

1.2 บทความเพื่อลงพิมพ์ในวารสารนานาชาติ

1.2.1 บทความซึ่งได้จัดส่งเพื่อการพิจารณาตีพิมพ์ในวารสารนานาชาติ 2 เรื่อง (ภาคผนวก ข) ได้แก่

- ก. Teachavorasinskun, S., Thongchim, P. and Lukkunaprasit, P. (2001). "Shear Modulus and Damping of Soft Bangkok Clays" *Canadian Geotechnical Journal* (provisionally accepted)

- ข. Pinkaew, T., Lukkunaprasit P. and Chatupote P. (2002). "Seismic Effectiveness of Tuned Mass Dampers for Damage Reduction of Structures", Engineering Structures, Journal of Earthquake, Wind and Ocean Engineering (provisionally accepted).

1.2.2 บทความซึ่งได้เตรียมต้นฉบับแล้ว (ภาคผนวก ค)

- ก. Lukkunaprasit, P., and Sittipunt, C. "Ductility Enhancement of Moderately Confined Concrete Tied Columns with Hook-Clips" (submitted to ACI Structural Journal)
- ข. Lukkunaprasit, P., and Thepmangkorn, J. "Load History Effect on Cyclic Behavior of R.C. Tied Columns. (to be submitted to J. Struct. Engrg., ASCE)
- ค. Wanitkorkul, A., and Lukkunaprasit, P., and Filiatrault, A. "Retrofit of Steel Moment-Resisting Frames Subjected to Long-Distance Earthquakes Considering Weld Fractures" (to be submitted to J. of Earthquake Engineering)

1.3 การเสนอผลงานวิจัยในที่ประชุมวิชาการนานาชาติ จำนวน 6 เรื่อง (ภาคผนวก ง)

- ก. Pinkaew, T. "Vibration Control of Structures under Harmonic Excitation using Semi-Active Mass Dampers", Proceedings of the 13th ASCE-Engineering Mechanics Conference, The John Hopkins University, Maryland, USA., 1999.
- ข. Lukkunaprasit, P. "Enhancement of Structural Performance for Moderate Seismic Risk Regions" Proceedings of the WG-1, Earthquake-Resisting Technologies for Civil Infrastructures, 3rd EQTAP Workshop, Manila, Philippines, 2000.
- ค. Lukkunaprasit, P. "An Innovative Hook-Clip for Performance of Tied Columns in Developing Countries", Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.
- ง. Sittipunt, C., and Wood, S. L. "Development Details to Improve the Cyclic Response of Slender Structural Walls", Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.
- จ. Lukkunaprasit, P., and Thepmangkorn, J. "Load History Effect on Cyclic Behavior of R.C. Tied Columns" Proceedings of the 14th KKNN Symposium on Civil Engineering, Kyoto, Japan, 2001.

- ฉ. Teachavorasinskun, S., and Amornwithayalax, T. "Elastic Shear Modulus of Clays During Undrained Shearing" Proceedings of the 14th KKNN Symposium on Civil Engineering, Kyoto, Japan, 2001.

1.4 การเสนอผลงานวิจัยในที่ประชุมวิชาการในประเทศ จำนวน 3 เรื่อง (ภาคผนวก จ) ได้แก่

- ก. พงษ์ธร จาญพจน์, ทศพล ปิ่นแก้ว (2543) การควบคุมการสั่นไหวของโครงสร้างอิลาสโตพลาสติคภายใต้การเคลื่อนที่ของฐานรองรับโดยใช้มวลหน่วงปรับค่า เอกสารประกอบการประชุมวิชาการวิศวกรรมโยธาแห่งชาติ ครั้งที่ 6 เพชรบุรี (STR61-66).
- ข. พิสิฐ ยิ่งมโนกิจ, ทศพล ปิ่นแก้ว (2543) การออกแบบมวลหน่วงปรับค่าแบบกึ่งแอคทีฟเพื่อการควบคุมการสั่นไหวของอาคาร เอกสารประกอบการประชุมวิชาการวิศวกรรมโยธาแห่งชาติ ครั้งที่ 6 เพชรบุรี (STR162-167).
- ค. ภัทรายุส ไวจรรยา, ทศพล ปิ่นแก้ว (2543) การป้องกันความเสียหายของอาคารสูงเมื่อรับแรงแผ่นดินไหวด้วยมวลหน่วงปรับค่า เอกสารประกอบการประชุมวิชาการวิศวกรรมโยธาแห่งชาติ ครั้งที่ 6 เพชรบุรี (STR168-173)

2. การจดทะเบียนสิทธิบัตร (ภาคผนวก ฉ)

ได้ดำเนินการเพื่อขอจดสิทธิบัตรสิ่งประดิษฐ์ “คลิปปิดขาของอเหล็กปลอก” กับกรมทรัพย์สินทางปัญญา (โดยผ่านทางสำนักงานทรัพย์สินทางปัญญา จุฬาลงกรณ์มหาวิทยาลัย) เมื่อ 28 ต.ค. 42 ขณะนี้กำลังรอการอนุมัติสิทธิบัตรอยู่

ชื่อสิทธิบัตร คลิปปิดขาของอเหล็กปลอก
ประเทศที่จดสิทธิบัตร ประเทศไทย

3. วิทยานิพนธ์ที่ได้ จำนวน 7 เรื่อง ได้แก่

- ผลของการเสริมเหล็กรับแรงเฉือนแบบต่าง ๆ ต่อพฤติกรรมแบบเป็นวัฏจักรของกำแพงคอนกรีตเสริมเหล็ก : พิชัย ภัทรรัตนกุล (2541)
- พฤติกรรมของเหล็กเสริมภายใต้แรงกระทำแบบวัฏจักร : วรพงษ์ จินช้าง (2541)
- คุณสมบัติทางด้านพลศาสตร์ของดินเหนียวอ่อนกรุงเทพฯ : พิพัฒน์ ทองฉิม (2542)
- ความต้านทานแผ่นดินไหวของอาคารคอนกรีตเสริมเหล็กที่ปราศจากความเหนียว ภัทรายุส ไวจรรยา (2543)
- การศึกษามวลหน่วงปรับค่าสำหรับควบคุมการสั่นไหวของโครงสร้างอันอิลาสติค พงษ์ธร จาญพจน์ (2543)

- การควบคุมการสั่นไหวของอาคารเนื่องจากแผ่นดินไหวโดยใช้มวลหน่วงปรับค่าแบบกึ่งแอกทีฟฟิลิรู ยิงมโนกิจ (2543)
- ผลของระดับแรงอัดตามแนวแกนและรูปแบบเหล็กปลอกเดี่ยวที่มีต่อความเหนียวและการสลายพลังงานของเสาคอนกรีตเสริมเหล็ก : จตุพล เทพมังกร (2544)

4. การสร้างนักวิจัยใหม่ จำนวน 12 คน

นายพิชัย ภักธรัตนกุล	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายวรพงษ์ จินช้าง	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายฟิลิรู ยิงมโนกิจ	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายพงษ์ธร จาญพจน์	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายภัทรายุส ไวจรรยา	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายจตุพล เทพมังกร	สำเร็จการศึกษาระดับปริญญาโทแล้ว
นายพงษ์สุร อังคณานูชาติ	กำลังศึกษาระดับปริญญาโท (คาดว่าจะสำเร็จในเดือนเม.ย. 45)
นายอัศวิน วาณิชย์ก่อกุล	กำลังศึกษาระดับปริญญาเอก
นายพิพัฒน์ ทองฉิม	กำลังศึกษาระดับปริญญาเอก
นายจารึก ถิระวงษ์	กำลังศึกษาระดับปริญญาเอก
นายภาสกร ชัยวิริยะวงศ์	กำลังศึกษาระดับปริญญาเอก
นายสมบูรณ์ เชียงฉิน	กำลังศึกษาระดับปริญญาเอก

5. การเผยแพร่ผลงาน

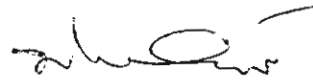
5.1 ผลงานวิจัยเกี่ยวกับ Hook-Clip ได้รับการเผยแพร่ใน (ดูภาคผนวก ข)

- หนังสือพิมพ์มติชน ฉบับวันเสาร์ที่ 20 พฤษภาคม 2543 หน้า 7
- หนังสือพิมพ์เดลินิวส์ ฉบับวันพุธที่ 24 พฤษภาคม 2543 หน้า 16
- นิตยสารข่าวช่าง ปีที่ 28 ฉบับที่ 338 หน้า 57-58
- นิตยสาร People ปีที่ 12 ฉบับที่ 134 หน้า 89
- รายการสุดสักรัชมใหม่ ในช่วงข่าวภาคค่ำ ช่อง 9 อสมท. วันที่ 27 พฤษภาคม 2543
- รายการรากแก้วแห่งปัญญา ช่อง 9 อสมท. เดือนมิถุนายน 2543
- รายการโลกสีน้ำเงิน ทาง itv วันที่ 27 ตุลาคม 2543
- Earthquake Hazard Center Newsletter, Victoria University of Wellington, New Zealand, Vol. 4, No.1, July 2000
- กระทรวงสาธารณสุข โดยกองแบบแผน ได้เห็นประโยชน์ของการประยุกต์สิ่งประดิษฐ์ดังกล่าว เพื่อเพิ่มความเหนียวให้แก่อาคารของกระทรวงที่จะก่อสร้างใน 10 จังหวัด ซึ่งกฎกระทรวงฉบับที่ 49 บังคับให้มีการออกแบบอาคารต้านแผ่นดินไหว ดังเอกสารที่แนบมา โดยจะกำหนดไว้ในแบบมาตรฐานของกระทรวงฯ

6. รางวัล (ภาคผนวก ข)

ศาสตราจารย์ ดร.ปณิธาน ลักคุณะประสิทธิ์ ได้รับรางวัลวิทยาศาสตร์และเทคโนโลยี มูลนิธิ
โทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย ประจำปี พ.ศ. 2544 ประเภทบุคคลที่มีผลงานทางด้านวิทยา-
ศาสตร์และเทคโนโลยีที่มีคุณภาพเป็นเลิศ ซึ่งได้รับการตีพิมพ์ในวารสารวิชาการที่มีการตรวจสอบคุณภาพอย่าง
เคร่งครัด ตลอดจนเป็นผลงานที่มีคุณค่าต่อสังคมในด้านการสร้างความก้าวหน้าทางวิชาการและในด้านศักยภาพ
ของการนำไปประยุกต์ใช้

ลงนาม



(ศาสตราจารย์ ดร.ปณิธาน ลักคุณะประสิทธิ์)

หัวหน้าโครงการวิจัยฯ

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บทความที่ตีพิมพ์ในวารสารนานาชาติ

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Inelastic buildings with tuned mass dampers under moderate ground motions from distant earthquakes

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SUMMARY

The effectiveness of tuned mass dampers (TMD) in vibration control of buildings was investigated under moderate ground shaking caused by long-distance earthquakes with frequency contents resembling the 1985 Mexico City (SCT) or the 1995 Bangkok ground motion. The elastic–perfectly plastic material behaviour was assumed for the main structure, with linear TMDs employed by virtue of their simplicity and robustness. The accumulated hysteretic energy dissipation affected by TMD was examined, and the ratio of the hysteretic energy absorption in the structure with TMD to that without it is proposed to be used, in conjunction with the peak displacement ratio, as a supplementary TMD performance index since it gives an indication of the accumulated damage induced in the inelastic structures. For the ground motions considered, TMD would be effective in reducing the hysteretic energy absorption demand in the critical storeys for buildings in the 1.8–2.8 s range. The consequence is reduction in damage of the buildings which would otherwise suffer heavy damage in the absence of TMD, resulting in economical restorability in the damage control limit state. This is of practical significance in view of the current trend toward performance-based design. Copyright © 2001 John Wiley & Sons, Ltd.

KEY WORDS: tuned mass dampers; moderate ground motion; distant earthquakes; elastic–perfectly plastic behaviour; hysteretic energy absorption ratio; performance index; damage reduction

1. INTRODUCTION

The effectiveness of tuned mass dampers (TMD) in vibration control of buildings for serviceability condition under wind load has been widely recognized. However, their usefulness in control of building motion undergoing inelastic deformation under earthquake excitations is not yet universally accepted. Kaynia *et al.* [1] made a preliminary assessment of the seismic effectiveness of TMD for elastic–perfectly plastic single-degree-of-freedom (SDOF) systems under an ensemble of historical earthquake motions recorded at western United States sites.

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With the use of TMDs small reduction in cumulative yielding ductility was observed whereas the reduction in ductility ratio was insignificant. Sladek and Klingner [2] studied the effect of TMD on seismic response of a 25-storey building equipped with a TMD with a generalized mass ratio of 0.026, designed in accordance with Den Hartog [3]. Under the 1940 El Centro earthquake, with the structural model undergoing stiffness degrading non-linear deformation, the peak tip displacement response was found to be almost unaffected, and yielding at the base of the core walls could not be eliminated despite the presence of the TMD. Chowdhury *et al.* [4] attempted to assess the effectiveness of TMDs as useful aseismic control devices by evaluating the inelastic response of the building considered by Sladek and Klingner [2] under various ground motions caused by western U.S. earthquakes with source distances of 8–43 km. For the range of TMDs considered with generalized mass ratio and TMD damping varying from 5–18 to 4–19 per cent, respectively, the peak-response of the building with TMD to that of the uncontrolled one was found to be 1.05–0.74, depending on the absorber mass and damping, with increasing peak response reduction for increasing generalized mass ratio in general. The significant statistical dispersion of the peak-response ratios led the investigators to be doubtful of the benefit of TMDs as aseismic control devices. Bernal [5] examined the behaviour of an inelastic SDOF structure-TMD system under the 1985 Mexico City (SCT) excitation which is narrow band with long duration. Reduction in the maximum displacement response was found to decrease with increasing inelastic excursions.

Not all the researchers, though, are not in favour of TMD. Jagadish *et al.* [6] investigated the seismic responses of a family of inelastic two-storey buildings with the top storey acting like a vibration absorber for horizontal ground motions. The study indicated that a reduction of 50 per cent in the ductility demand on the lower storey was possible under the 1952 Taft earthquake ground motion. Jara and Aguiñiga [7] studied the effectiveness of elastic TMDs in the vibration control of elasto-plastic SDOF systems under the Mexico City (SCT) ground motion. It was found that significant reduction in the maximum responses of 25–40 per cent could be obtained with mass and damping ratios of 4–6 per cent when the ductility demand of the main system was equal to 4. Recently, Soto-Brito and Ruiz [8] investigated the effectiveness of TMD for a 22-storey reinforced-concrete frame under moderate and high-intensity ground motions. The study indicated that TMDs could be efficient in reducing the interstorey drifts of buildings undergoing small nonlinearity caused by moderate earthquakes. However, the efficiency of the TMDs would be much reduced under high-intensity ground motions.

From the literature review it may be observed that most studies, except the ones by Bernal [5], Jara *et al.* [7] and Soto-Brito and Ruiz [8], focused on ground motions which were predominantly broad band high-frequency excitations of short duration. Therefore, the findings obtained cannot be generalized for the case where the excitation is narrow band and of long duration, which is characteristic of the motion induced by long-distance earthquakes. Since many big cities in moderate seismic risk regions, such as Bangkok and Singapore, are known to be situated sufficiently far away from major active faults, the effect of long-distance earthquakes, which tend to produce resonance-like motion in medium to high-rise buildings, is of concern. The harmonic-like ground motion from long-distance earthquakes coupled with long excitation duration leads to a favourable condition for the TMDs in vibration control of buildings. While several studies exist which were directed toward high-intensity earthquake motions, little attention has been given to moderate ones, for which TMD should be of advantage in reducing damage in buildings which would otherwise be heavily damaged if the TMDs were not equipped. Furthermore, improved performance of buildings is also of increasing

importance in view of the current trend toward performance-based design which, depending on the performance objectives, might call for economical restorability under the damage control limit state. Due to the use of the displacement-based indices in previous studies, the effect of cumulative damage in the main structure due to cyclic deformation of the inelastic members cannot be accounted for in the evaluation of the TMD effectiveness. Therefore, a detailed examination of the accumulated energy dissipation affected by TMD is needed, and the ratio of the hysteretic energy absorption in the structure with TMD to that without it is proposed to be used, in conjunction with the peak displacement ratio, as a supplementary TMD performance index, since it gives an indication of the accumulated damage induced in the inelastic structures.

This study focused on moderate seismic risk regions. The effectiveness of TMD in reducing damage in buildings undergoing inelastic deformation under ground shaking pertinent to moderate seismic zones is demonstrated through numerical examples. For simplicity, the elastic-perfectly plastic behaviour was assumed for the main structure which was modelled as an N -storey shear building, and geometric non-linearity was not considered. Linear TMDs were employed by virtue of their simplicity and robustness.

2. EQUATIONS OF MOTION

Although the formulation of the problem is straight forward [9], an alternative approach is proposed which can be readily implemented to modify the existing non-linear dynamic analysis programmes to solve structural systems incorporating TMDs without much difficulty.

Consider an N -storey shear building with a TMD attached at the top as shown in Figure 1. Denoting $\mathbf{u}(t)$ as the lateral floor displacements relative to the base and $u_g(t)$ as the horizontal ground displacement at any time t , one may visualize the TMD sub-system as being subjected to the excitation of the top floor $\ddot{u}_N(t) + \ddot{u}_g(t)$, and hence arrive at the equation of motion of the TMD sub-system as

$$m\ddot{z}(t) + c\dot{z}(t) + kz(t) = -m(\ddot{u}_g(t) + \ddot{u}_N(t)) \quad (1)$$

where $z(t)$ is the relative displacement of the TMD mass with respect to the top floor and m , c , k are the mass, damping coefficient and stiffness of the added mass, respectively. The main structure sub-system, i.e. the building with TMD detached, is thus subjected to the ground acceleration $\ddot{u}_g(t)$ as well as an interactive force $P_N(t)$ induced by the motion of the TMD at the top floor, assumed to act in the horizontal direction, where

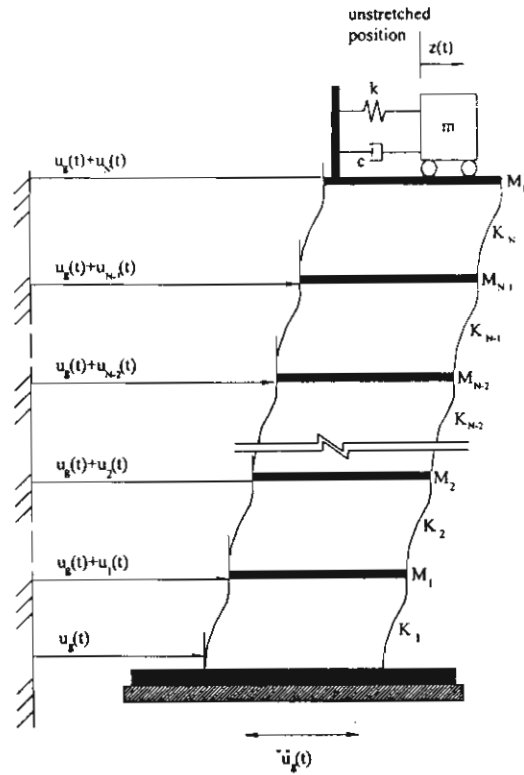
$$P_N(t) = kz + c\dot{z} \quad (2)$$

The equations of motion of the main structure sub-system can thus be simply written as

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{F}_S(\mathbf{u}, \dot{\mathbf{u}}) = \mathbf{F}_g(t) + \mathbf{P}(t) \quad (3)$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices of the main structure and $\mathbf{F}_S(\mathbf{u}, \dot{\mathbf{u}})$ is the vector of resisting forces in the columns, assumed to follow the elastic-perfectly plastic hysteresis law. $\mathbf{F}_g(t)$ is the effective force due to ground motion given by

$$\mathbf{F}_g(t) = -\mathbf{M}\mathbf{e}_g\ddot{u}_g(t) \quad (4)$$

Figure 1. Model of an N -storey shear building—TMD system.

in which e_g is a displacement influence coefficient vector associated with a unit support displacement, which is a column vector with elements of 1's in the case of a shear building; and

$$P(t) = [0, \dots, P_N(t)]^T \quad (5)$$

It is worth noting that the formulation given can be applied to a general MDOF system with proper treatment of the interactive force and effective force due to ground motion.

Equations (1)–(3) can be combined to yield the equations of motion governing the vibration of the $(N+1)$ DOF main structure-TMD system. For the special case of a SDOF main structure, the resulting equations of motion are

$$(M_1 + m)\ddot{u}_1(t) + C_1\dot{u}_1(t) + F_{s1}(u_1, \dot{u}_1) = -(M_1 + m)\ddot{u}_g(t) - m\ddot{z}(t) \quad (6)$$

and

$$m\ddot{z}(t) + c\dot{z}(t) + kz(t) = -m(\ddot{u}_g(t) + \ddot{u}_1(t)) \quad (7)$$

which are usually found in the literature.

3. AN ITERATIVE SCHEME

We propose to work with the sub-systems as governed by Equations (1) and (3) rather than conventionally solving the combined system of $(N + 1)$ DOF. The associated incremental equations of motion can be easily derived by considering the states at times t_i and t_{i+1} . Thus,

$$\mathbf{M}\Delta\ddot{\mathbf{u}}_i + \mathbf{C}\Delta\dot{\mathbf{u}}_i + \Delta\mathbf{F}_{Si} = \Delta\mathbf{F}_{gi} + \Delta\mathbf{P}_i \quad (8)$$

$$m\Delta\ddot{z}_i + c\Delta\dot{z}_i + k\Delta z_i = -m(\Delta\ddot{u}_{gi} + \Delta\ddot{u}_{Ni}) \quad (9)$$

where

$$\Delta\mathbf{u}_i = \mathbf{u}(t_{i+1}) - \mathbf{u}(t_i) \quad (10a)$$

$$\Delta\mathbf{F}_{Si} = \mathbf{F}_s(\mathbf{u}_{i+1}, \dot{\mathbf{u}}_{i+1}) - \mathbf{F}_s(\mathbf{u}_i, \dot{\mathbf{u}}_i) \quad (10b)$$

$$\Delta\mathbf{F}_{gi} = -\mathbf{M}\mathbf{e}_g(\ddot{u}_g(t_{i+1}) - \ddot{u}_g(t_i)) \quad (10c)$$

$$\Delta\mathbf{P}_i = [0, \dots, \Delta P_{Ni}]^T \quad (10d)$$

$$\Delta P_{Ni} = c(\dot{z}(t_{i+1}) - \dot{z}(t_i)) + k(z(t_{i+1}) - z(t_i)) \quad (10e)$$

$$\Delta z_i = z(t_{i+1}) - z(t_i) \quad (10f)$$

$$\Delta\ddot{u}_{gi} = \ddot{u}_g(t_{i+1}) - \ddot{u}_g(t_i) \quad (10g)$$

$$\Delta\ddot{u}_{Ni} = \ddot{u}_N(t_{i+1}) - \ddot{u}_N(t_i) \quad (10h)$$

For any time step t_i – t_{i+1} , with incremental ‘effective seismic force’ acting on the system, the main structure sub-system would first be assumed to receive no ‘incremental’ interactive force from the TMD. The ground motion would induce an incremental motion $(\Delta\mathbf{u}_i, \Delta\dot{\mathbf{u}}_i, \Delta\ddot{\mathbf{u}}_i)$ to the main structure. This incremental motion, which is obtained by solving Equation (8), is then introduced to further excite the TMD, and the induced incremental motion of the TMD is obtained by solving Equation (9). The incremental interactive force, $\Delta P_{Ni}(t)$ at the main structure–TMD interface is then obtained and fed back to the main structure, and the iteration process is carried out by iteratively solving Equations (8) and (9) successively until the incremental displacements approach converged values (within an acceptable tolerance).

The incremental equations of motion can be conveniently solved using the well-known Newmark method [10, 11]. For the example problems given herein, which involve only a small number of DOF, the linear acceleration method was chosen. The integration time step employed was in the order of 0.05 times the smallest natural period of vibration of the structure, which was much smaller than that required to ensure a stable numerical solution.

4. TMD EFFECTIVENESS INDEX

The most commonly used index to measure the effectiveness of tuned mass dampers is the peak response ratio which is the ratio of the peak response of the maximum relative

Table I. Properties of the main structural model and TMD ($T_1 = 2.0$ s).

Storey no.	Storey mass (Mg $\times 10^4$)	Stiffness (kN/m $\times 10^5$)	Yield strength (kN $\times 10^4$)
5	0.28	3.70	2.94
4	0.48	4.31	3.67
3	0.53	4.93	4.40
2	0.57	5.55	5.14
1	1.18	6.16	5.87

Note: TMD mass = 752 Mg; stiffness = 6.60×10^3 kN/m; $\xi = 14.6\%$.

displacement of the building with and without TMD. Obviously, this measure cannot account for the effects of accumulation of damage that occurs in an elastic-plastic system under cyclic loading. Therefore, the effectiveness of TMD cannot be solely judged from the peak response ratio. Since cumulative damage is related to the hysteretic energy absorbed during cyclic loading [12], it is quite natural to adopt a hysteretic energy absorption ratio, ρ_E , in conjunction with the peak displacement ratio, as a measure of the effectiveness of TMD, i.e.

$$\rho_E = \frac{E_{h,TMD}}{E_{h,0}} \quad (11)$$

where $E_{h,TMD}$ and $E_{h,0}$ are the accumulated hysteretic energy absorption in the system with and without TMD, respectively.

5. NUMERICAL STUDIES

5.1. Building with fundamental period close to predominant period of ground motion

To examine the usefulness of TMD in improving the performance of buildings under long-distance earthquakes, a tall building in Bangkok whose natural period (T_1) was 2.0 s was first taken as a case study. For simplicity, the building (without TMD) was idealized as a 5-DOF elasto-plastic shear building. The storey stiffness and yield strength were assumed to vary linearly with height, with the values of the top storey being 60 and 50 per cent of those in the bottom, respectively. The bottom storey yield strength was taken as 1/4 of the elastic base shear which would result if the building were subjected to the scaled Mexico ground motion, to be defined in the next section. The damping ratio of the building was taken as 2 per cent for each mode of vibration. The TMD damping ratio and tuning frequency were based on the Den Hartog method [3] with a moderate mass ratio of 6 per cent selected for practical purposes. Table I lists the relevant properties of the main structural model and the TMD.

The ground motions considered were the 1985 Mexico city (SCT) record and the 1995 Bangkok accelerogram recorded at the base of the Baiyoke tower whose fundamental period of vibration is 2.5 s. The peak ground accelerations (PGA) were scaled to the value of 0.10 times the acceleration due to gravity (g) to represent moderate ground shaking. In view of the long duration of excitation together with excessive damage in Mexico City caused by the 1985 Mexico earthquake, the scaled ground motions can be regarded as representatives of strong

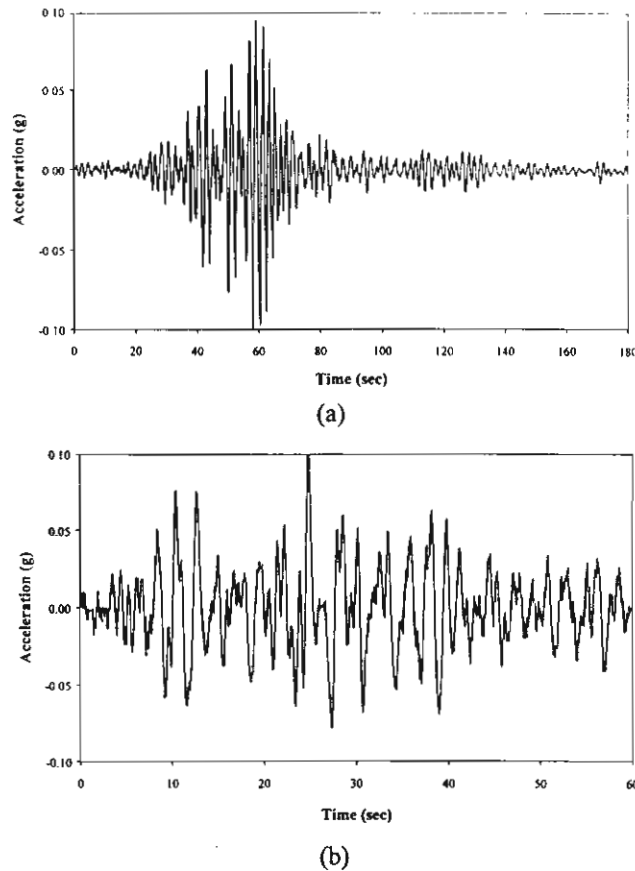


Figure 2. Scaled ground motion records: (a) 1985 Mexico city (SCT); (b) 1995 Bangkok.

ground shaking in a moderate seismic risk area. In fact the PGA of $0.1g$ was about twice of that from the SCT-89 record considered by Soto-Brito and Ruiz [8] as moderate ground motion. Figure 2 shows the ground motion records, and the associated response spectra are shown in Figure 3. As is widely known, the Mexico record is significantly narrow band with a predominant frequency of 0.5 Hz. On the other hand, the Bangkok record, which resulted from an earthquake event whose epicentre was about 900 km from Bangkok, contains not only the predominant low-frequency component with 0.44 Hz, but also significantly higher and lower frequency components as well.

Figures 4–7 illustrate the displacement and base shear responses. Also plotted in the figures are the results for the uncontrolled building. It may be observed that for the scaled Mexico ground motion, the peak displacement for the building with TMD was reduced by a small amount of 12 per cent from that of the uncontrolled building, which was deformed to the plastic range as many as 12 cycles in the bottom storey. It is interesting to note, however, that the number of yielding cycles in the ground floor columns was reduced significantly by

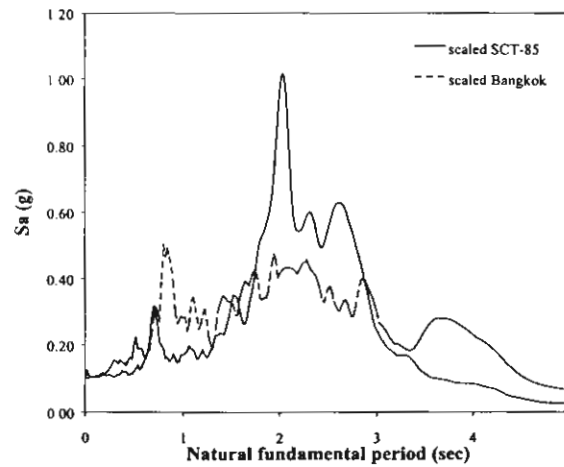


Figure 3. Response spectra of scaled Mexico and Bangkok ground motions (0.10g PGA, 2 per cent critical damping).

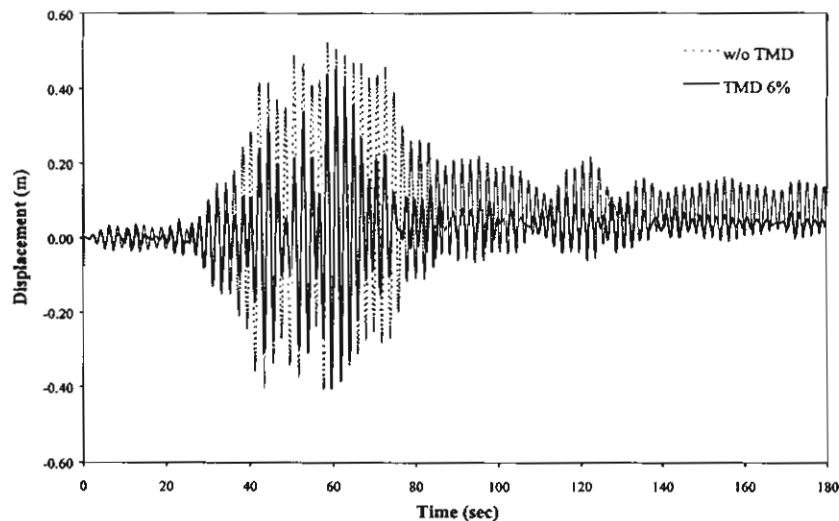


Figure 4. Displacement response of elasto-plastic shear frame with $T_1 = 2.0$ s to scaled Mexico (SCT-85) ground motion.

about 70 per cent with the aid of TMD (Figure 5). Clearly the building with TMD would suffer much less damage than the one without it. This could be best quantified by means of the accumulated hysteretic energy absorption of the bottom storey since it accounted for about 80 per cent of the total hysteretic energy absorption in the building. The dissipated energy in

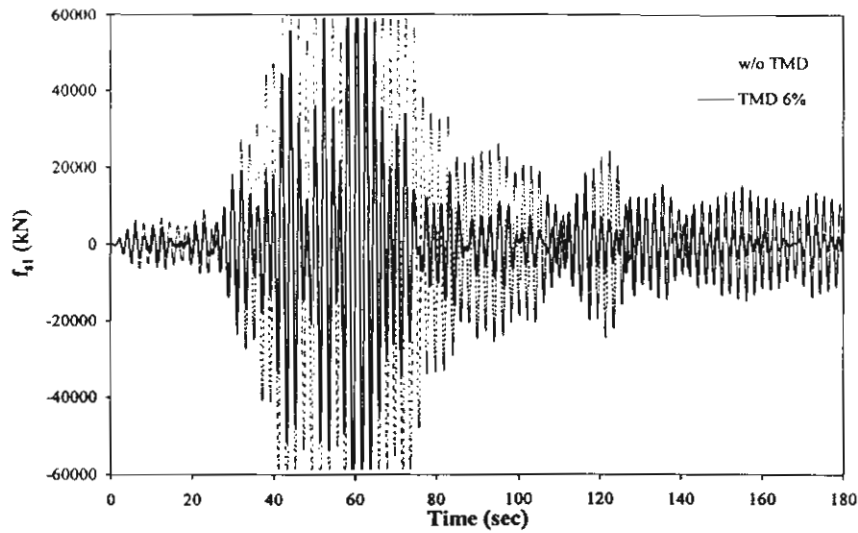


Figure 5. Base shear response of elasto-plastic shear frame with $T_1 = 2.0$ s to scaled Mexico (SCT-85) ground motion.

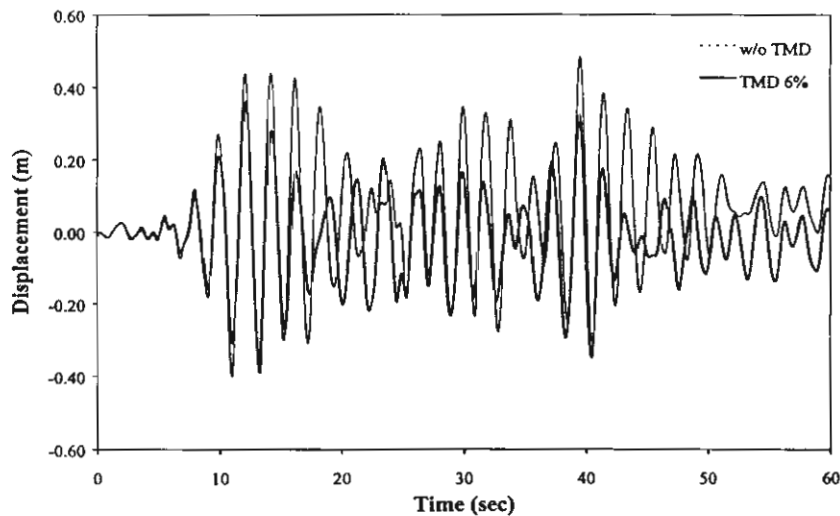


Figure 6. Displacement response of elasto-plastic shear frame with $T_1 = 2.0$ s to scaled Bangkok ground motion.

the ground floor columns of the controlled building was found to be only 20 per cent of that in the uncontrolled one.

For the Bangkok ground motion, the peak displacement reduction was almost 20 per cent, and the reduction in the hysteretic energy absorption in the ground floor columns was as much

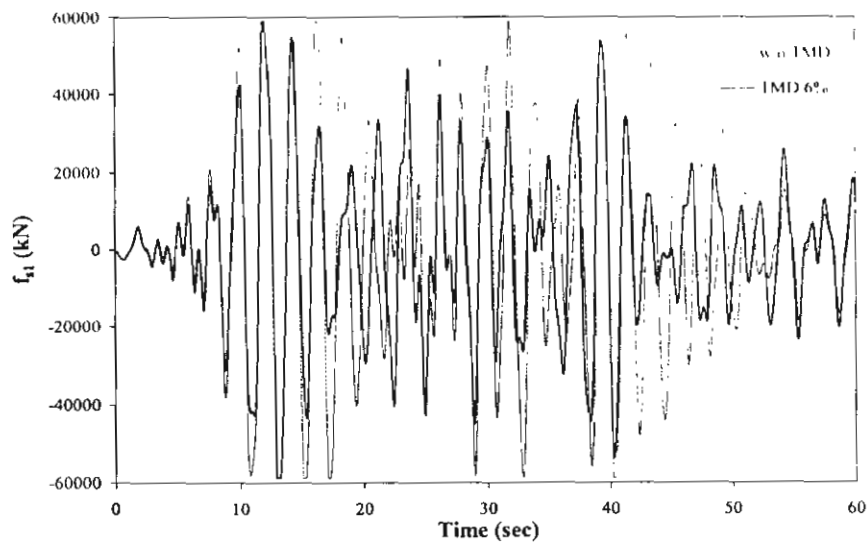


Figure 7. Base shear response of elasto-plastic shear frame with $T_1 = 2.0$ s to scaled Bangkok ground motion.

as 90 per cent. An inspection of the base shear response history reveals that the ground floor columns experienced only 1 cycle of yielding in the case of the Bangkok ground motion, compared to 4 for the case of the scaled Mexico earthquake. The hysteretic energy absorption for the Bangkok excitation was only about 10 per cent of that caused by the scaled Mexico earthquake. Hence the detuning effect was much less in the former case, leading to more effectiveness in TMD performance.

It is evident that TMD is beneficial for the building concerned, and its effectiveness can be attributed to the long duration and harmonic-like nature of the input excitations. Such ground motions, typical of long-distance earthquakes, tend to excite the building in resonance when the natural period of the building is close to the predominant period of the ground motion, and it takes some time before the peak response is attained, thereby allowing the TMD to play its role.

5.2. TMD performance with variation of building natural periods

It is interesting to investigate how TMD performs for buildings whose fundamental periods of vibration are different from the predominant period of the ground motion. A family of buildings with periods of 1.7, 1.8, 2.15, 2.27, 2.6, 2.8 and 3.0 s were subjected to the scaled Mexico and Bangkok excitations. The 2.27 s period was included to match the predominant period of the

Table II. Properties of the building-TMD system with various periods.

Storey no.	Storey mass (Mg) $\times 10^4$						
	$T_1 = 1.70$ s	$T_1 = 1.80$ s	$T_1 = 2.15$ s	$T_1 = 2.27$ s	$T_1 = 2.60$ s	$T_1 = 2.80$ s	$T_1 = 3.00$ s
5	0.16	0.17	0.28	0.28	0.25	0.27	0.29
4	0.28	0.30	0.48	0.48	0.44	0.47	0.50
3	0.31	0.33	0.53	0.53	0.48	0.52	0.55
2	0.33	0.35	0.57	0.57	0.51	0.55	0.59
1	0.69	0.74	1.18	1.18	1.06	1.14	1.23
K_1 (kN/m) $\times 10^5$	5.04	4.76	5.33	4.78	3.29	3.06	2.86
F_{y1} (kN) $\times 10^4$	1.31	2.01	4.64	4.70	3.37	2.71	1.41
m_{TMD} (Mg)	444	471	752	752	680	732	784
K_{TMD} (kN/m) $\times 10^3$	5.40	5.10	5.71	5.12	3.53	3.28	3.06
ξ_{TMD} (%)	14.6	14.6	14.6	14.6	14.6	14.6	14.6

Bangkok earthquake excitation. Again, the buildings were equipped with the Den Hartog TMDs with a generalized mass ratio of 6 per cent. The building and TMD properties are listed in Table II. As in the previous case, the bottom storey shear strengths were assigned values of 1/4 of the elastic solution based on the scaled Mexico ground motion, except for the 2.15 and 2.27 s period buildings whose base shear strengths were adjusted as described in the next section. The variations in storey stiffnesses and shear strengths with height were in the same manner as in the previous case.

As is evident in Figure 8, within the building periods of 1.8–2.8 s, the peak displacement ratio fluctuates in a narrow range of 0.78–1.0 and 0.71–0.97 for the scaled Mexico and Bangkok ground motions, respectively, while the hysteretic energy absorption ratio for the bottom storey varies between 0.10 and 0.61 as depicted in Figure 9. The TMD performance is close to optimum when the building period is in the neighbourhood of the predominant period of the Mexico ground motion. Elsewhere, the detuning effect will be more pronounced leading to deterioration in TMD performance. It may be observed that over the building period range of 1.8–2.8 s, the reduction in the dissipated energy is 40 per cent or more even though there is negligible reduction in the peak displacement response. The number of yielding excursions in the bottom storey is also reduced by more than 40 per cent in general. The results clearly indicate that TMD can be effective in reducing the inelastic deformation, with the consequence of less damage, for buildings in the 1.8–2.8 s period range when they are subjected to motions with characteristics similar to the Mexico City (SCT) or Bangkok earthquakes.

5.3. Fine tuning of storey yield strength

As mentioned in the last section, the storey yield strength was based on the elastic response which resulted from the scaled Mexico ground motion. The elastic base shear attained a peak value at the building period of close to 2 s and fell quite sharply. With the bottom storey yield strength taken as 1/4 of the elastic solution, the values would be 58,700 and 34,100 kN for

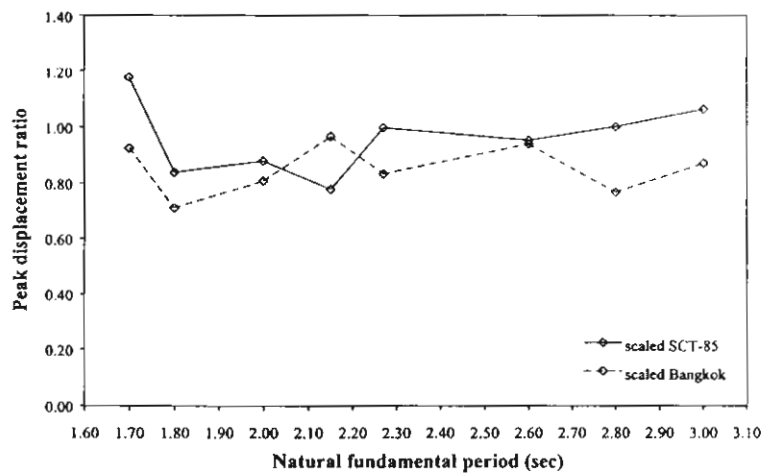


Figure 8. Variation of peak displacement ratio with natural period of building.

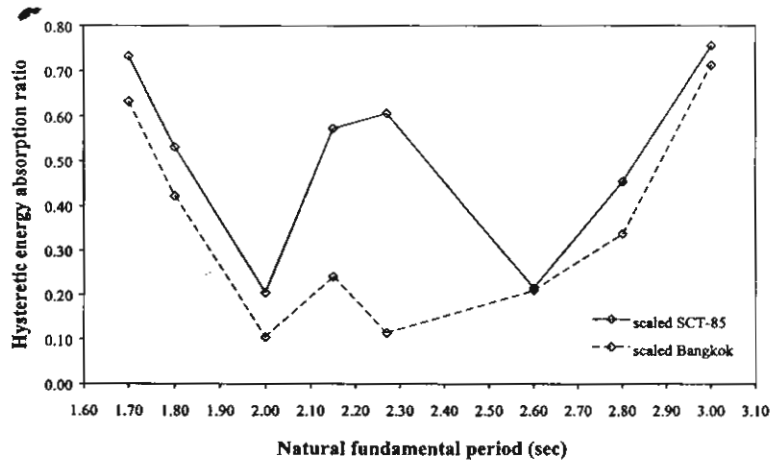


Figure 9. Variation of hysteretic energy absorption ratio with natural period of building.

the buildings with periods of 2.0 and 2.15 s, respectively. One may observe that the bottom storey yield strength decreased by as much as 42 per cent with an increase in building period of only 7.5 per cent. The sharp reduction was due to the effect of narrow band excitation. With the bottom storey yield strength taken as calculated, the peak displacement ratio for the 2.15 s period building was found to be 1.17. However, with the base shear strength of this building taken as the average of the values of the two buildings, the peak displacement ratio was reduced significantly from 1.17 to 0.78. In practice, a change in building period of

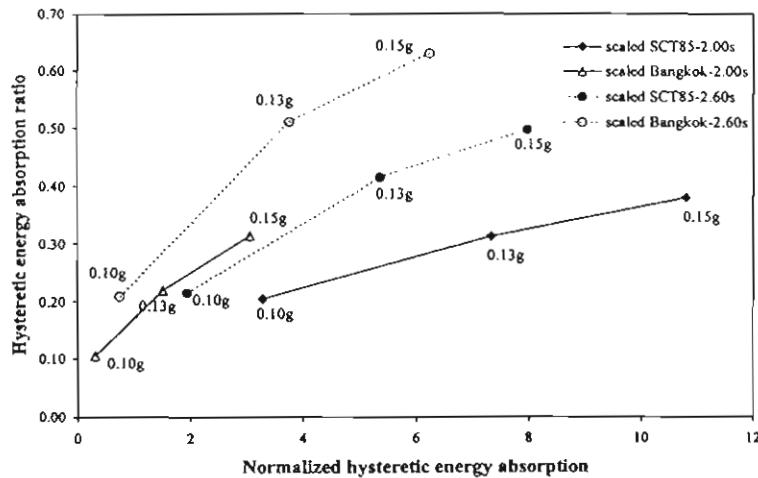


Figure 10. Hysteretic energy absorption ratio as function of normalized hysteretic energy absorption in the bottom storey for building periods of 2 and 2.6 s.

7.5 per cent is considered insignificant, and it would not be sensible to design the 2.15 s period building with only 60 per cent of the storey yield strength of the 2 s period building. Therefore, it would be appropriate to fine tune the storey yield strength of the building so that better performance of the building is achieved.

5.4. Damage reduction by TMD for moderate ground shaking

To examine further the benefit of TMD, the performance of two buildings was investigated under increasing peak ground accelerations. The fundamental periods of the buildings considered were 2.0 and 2.6 s. Figure 10 depicts the hysteretic energy absorption in the bottom storey of the main structure-TMD system as a function of the normalized hysteretic energy absorption ratio, with PGA values shown in the figure. The normalized hysteretic energy absorption, β_E , is defined as

$$\beta_E = \frac{E_{h1,TMD}}{f_{y1}\Delta_{y1}} \quad (12)$$

in which $E_{h1,TMD}$ is the accumulated hysteretic energy absorption in the bottom storey of the building with TMD, and f_{y1} and Δ_{y1} are the storey yield strength and relative yield displacement of the bottom storey, respectively. The normalized hysteretic energy absorption could be generalized to include all members, but only the bottom storey was considered here since it accounted for a large portion of the hysteretic energy for the buildings considered.

As mentioned earlier, the earthquakes considered in this study were intended to represent rare events in moderate seismic risk regions. The buildings without TMD, though capable of

withstanding the ground shaking, would suffer significant damage. Depending on the design for ductility as well as characteristics of the hysteresis loops, a value of 15 or more may be the ultimate limit of the normalized hysteretic energy for moderate level ductility, which needs to be calibrated against experimental results. Taking a value of 15 as the limit, for instance, the 2 s period building without TMD would be on the verge of collapse under the scaled Mexico ground motion with PGA of 0.10g. However, with the contribution of TMD, the critical columns in the building-TMD system are now called upon to dissipate only 20 per cent of the hysteretic energy of the system without TMD (see Figure 10), while the ductility demand is also reduced slightly. Consequently, the building with TMD would survive the earthquake with much less damage. Likewise, the 2.6 s period building, which would be very severely damaged under a PGA of 0.13g in the absence of TMD, would survive with moderate damage with the help of TMD, and hence it would be less costly to repair. This is of practical significance in view of the current trend toward performance-based design.

As already found by previous researchers, TMD deterioration in performance is again observed with increasing hysteretic energy dissipation.

6. CONCLUSIONS

The effectiveness of TMD in vibration control of buildings undergoing elasto-plastic deformation was investigated under moderate ground shaking induced by long-distance earthquakes with frequency contents resembling the 1985 Mexico City (SCT) or the Bangkok ground motion. The following conclusions can be drawn from this study:

1. Unlike in an elastic system, the commonly used peak displacement ratio cannot give a good indication of the TMD effectiveness for an elasto-plastic building, since significant reduction in the dissipated energy may result in a building with TMD even though only a small reduction in the displacement response is effected by the presence of the TMD. The hysteretic energy absorption ratio should be supplemented as another performance index because it reflects the amount of dissipated energy in the yielding elements in both the controlled and uncontrolled buildings, hence the extent of damage induced.
2. For the ground motions considered which have predominant periods in the range of 2–2.3 s, the TMD can be employed to effectively reduce the hysteretic energy absorption demand in the critical storeys for buildings in the 1.8–2.8 s range. For a building whose bottom storey yield strength is taken as 1/4 of the elastic value resulting from the scaled Mexico ground motion, with appropriate fine tuning near the resonant frequency, this reduction can be as large as 40 per cent or more.
3. For a well designed system, the TMD can be used to advantage to reduce the damage of buildings which would otherwise suffer heavy damage in the absence of TMD, resulting in economical restorability in the damage control limit state.
4. Although optimizing of system characteristics was beyond the scope of this paper, a case study was carried out in which fine tuning of the base yield strength was found to be advantageous in improving the performance of the building.

Effectiveness of semi-active tuned mass dampers under harmonic excitation

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Effectiveness of semi-active tuned mass dampers under harmonic excitation

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Abstract

The control effectiveness of a semi-active tuned mass damper (STMD) with variable damping under harmonic excitation is studied. An optimal control law governing the variation of damping of the damper is derived using the optimal control theory. A situation in which the damping can be varied within a certain range is included. Employing the numerical technique, the responses of a single-degree-of-freedom (SDOF) structure coupled with an STMD are investigated. The control effectiveness of the STMD is evaluated by comparing the structure's transient and steady-state responses with those of the structure coupled with a tuned mass damper (TMD). It is found that, under harmonic excitation, the vibration suppression by an STMD is significantly superior to that of a conventional passive TMD in both transient and steady-state responses. This improvement is equivalent to an increase of the TMD's mass by about four times. © 2001 Elsevier Science Ltd. All rights reserved.

Keywords: Harmonic excitation; Tuned mass damper; Semi-active mass damper; Vibration control

1. Introduction

To minimize vibration, reduction of the external disturbance to a system is always the preferred countermeasure, but, in many cases, this may not be possible. Modification of the system to avoid resonance may entail significant redesign. Furthermore, it would be difficult to apply to existing structures. Thus, vibration dampers, which can be simply attached to the existing system to reduce the vibration without drastically altering the original system, have been developed.

The passive tuned mass damper (TMD) is undoubtedly a simple, inexpensive and reliable means to suppress the undesired vibrations of systems. However, its performance is limited because of fixed damper parameters. The very narrow band of suppression frequency, the ineffective reduction of non-stationary vibration, and the sensitivity problem due to detuning are the inherent limitations of the passive TMD [1,2]. It has been estab-

lished that the addition of an active force actuator can considerably enhance the TMD's effectiveness [3]. This passive TMD with an actuator is known as the active tuned mass damper or hybrid mass damper.

Active tuned mass dampers (ATMDs) have been studied extensively for the last two decades [4]. It has been reported that, in all cases, the ATMD can provide better suppression performance of structural vibrations than the passive TMD. Although the ATMD demonstrates superior performance than the best possible passive system, the active system is more costly, more complex and needs careful maintenance. This makes it less reliable than the passive system. Thus the implementation of ATMDs has been limited to certain cases. Recognizing both the performance benefits as well as the limitations of active systems, the concept of a semi-active mass damper has been introduced.

As an extension of the semi-active damping concept that has been successfully applied to a broad class of vehicle vibration isolation problems [5–7], a semi-active tuned mass damper (STMD) with variable damping has been proposed for structural vibration control. For this kind of damper, a passive damping device is replaced by an adjustable damping device such as a variable-orifice

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hydraulic damper or a device using magnetorheological fluid. Since only a small amount of active energy is required to modulate the damping of such devices, the need for a large energy supply as required by the ATMD is eliminated. Hence, it provides an extremely promising alternative to passive and active tuned mass dampers. Hrovat et al. [8] investigated the benefits of using an STMD for the suppression of wind-induced vibration. Assuming a specific type of wind loading, similar performance for the STMD and ATMD was reported. Hidaka et al. [9] conducted an experimental study of an STMD coupled with a three-storey building model under support ground motion. The objective was to reduce the structural response at several frequencies of vibration with a single mass damper. Using their proposed neural network control technique, they concluded that the vibration response at the first and second natural frequencies of the structure could be reduced. However, there is no explicit study on the control performance of STMDs under harmonic excitation, which provides very useful information for a wide range of control applications.

The purpose of this paper is to investigate the control characteristics of the STMD for vibration suppression in harmonically excited structures. Employing a numerical technique, the transient and steady-state responses of structures coupled with a TMD and with an STMD are compared and discussed.

2. Optimal semi-active mass damper

2.1. General solution

A model representing a structure attached to the semi-active tuned mass damper (STMD) is shown in Fig. 1.

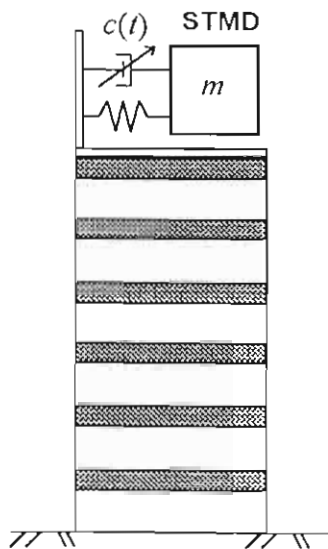


Fig. 1. Structure with STMD system.

The equations of motion are written in the following state-space description as

$$\dot{\mathbf{x}}(t) = \mathbf{A}\mathbf{x}(t) + \mathbf{B}u_{\text{semi}}(t), \quad (1)$$

where $\mathbf{x}(t)$ is a system state vector, $\mathbf{A}$ is a system matrix, $\mathbf{B}$ is a control contribution vector and

$$u_{\text{semi}}(t) = \mathbf{x}^T(t)\mathbf{T}c(t) \quad (2)$$

is a semi-active control force which is the product of a state vector, $\mathbf{x}(t)$, a transformation vector, $\mathbf{T}$, and a variable damping coefficient, $c(t)$. The design objective is to find the optimal damping control $c(t)$ that minimizes the quadratic performance index [10]

$$J_{\text{semi}} = \int_0^{t_f} [\mathbf{x}^T(t)\mathbf{Q}\mathbf{x}(t) + ru_{\text{semi}}^2(t)] dt + \mathbf{x}^T(t_f)\mathbf{P}_f\mathbf{x}(t_f) \quad (3)$$

subjected to the dynamic constraints (1) and (2). In Eq. (3), $\mathbf{Q}$ is a positive semi-definite matrix, r is a positive constant, $\mathbf{P}_f$ is a positive semi-definite matrix and t_f is the final time of a control interval.

Similar to the determination of an optimal semi-active vehicle suspension system [10], it is convenient to first consider the corresponding optimal solution for the active system case. Then the optimal solution for the semi-active case is determined by minimizing the variation of performance index, which deviates from its active optimum due to the introduction of a semi-active system.

Replacing $u_{\text{semi}}(t)$ in Eq. (1) by $u_{\text{active}}(t)$, the equations of motion for the system become

$$\dot{\mathbf{x}}(t) = \mathbf{A}\mathbf{x}(t) + \mathbf{B}u_{\text{active}}(t). \quad (4)$$

In writing the above equation, $u_{\text{active}}(t)$ is an active control force which can be arbitrarily assigned without constraint (2) as in a semi-active case. The fully active optimal control, $u_{\text{active}}(t)$, which minimizes the quadratic performance

$$J_{\text{active}} = \int_0^{t_f} [\mathbf{x}^T(t)\mathbf{Q}\mathbf{x}(t) + ru_{\text{active}}^2(t)] dt + \mathbf{x}^T(t_f)\mathbf{P}_f\mathbf{x}(t_f), \quad (5)$$

is known to be [11]

$$u_{\text{active}}(t) = -r^{-1}\mathbf{B}^T\mathbf{P}(t)\mathbf{x}(t), \quad (6)$$

where the co-state matrix, $\mathbf{P}(t)$, is obtained from

$$\begin{aligned} \dot{\mathbf{P}}(t) &= -\mathbf{A}^T\mathbf{P}(t) - \mathbf{P}(t)\mathbf{A} - \mathbf{Q} + \mathbf{P}(t)\mathbf{B}r^{-1}\mathbf{B}^T\mathbf{P}(t); \quad \mathbf{P}(t_f) \\ &= \mathbf{P}_f \end{aligned} \quad (7)$$

To find the performance deviation due to the introduction of a semi-active constraint (2), we consider the time derivative of the term $\mathbf{x}^T(t)\mathbf{P}(t)\mathbf{x}(t)$, and obtain

$$-\frac{d}{dt}(\mathbf{x}^T(t)\mathbf{P}(t)\mathbf{x}(t)) = -\dot{\mathbf{x}}^T(t)\mathbf{P}(t)\mathbf{x}(t) - \mathbf{x}^T\dot{\mathbf{P}}(t)\mathbf{x}(t) \quad (8)$$

$$-\mathbf{x}^T(t)\mathbf{P}(t)\dot{\mathbf{x}}(t).$$

Using the identity

$$r(u_{\text{semi}}(t) - u_{\text{active}}(t))^2 = ru_{\text{semi}}^2(t) + \mathbf{x}^T(t)\mathbf{P}(t)\mathbf{B}r^{-1}\mathbf{B}^T\mathbf{P}(t)\mathbf{x}(t) + 2\mathbf{x}^T(t)\mathbf{P}(t)\mathbf{B}u_{\text{semi}}(t) \quad (9)$$

and consulting Eqs. (4) and (7), the integration of Eq. (8) from time $t=0$ to $t=t_f$ can be written as

$$\begin{aligned} \mathbf{x}^T(0)\mathbf{P}(0)\mathbf{x}(0) - \mathbf{x}^T(t_f)\mathbf{P}_f\mathbf{x}(t_f) &= \int_0^{t_f} [\mathbf{x}^T(t)\mathbf{Q}\mathbf{x}(t) \\ &+ ru_{\text{active}}^2(t)] dt - \int_0^{t_f} [r(u_{\text{semi}}(t) - u_{\text{active}}(t))^2] dt. \end{aligned} \quad (10)$$

Taking the limit as $t_f \rightarrow \infty$ and assuming $\mathbf{P}_f = 0$, we can now relate the performance index of the general semi-active system to that of the corresponding active system as

$$J_{\text{semi}} = J_{\text{active}} + \int_0^{\infty} [r(u_{\text{semi}}(t) - u_{\text{active}}(t))^2] dt, \quad (11)$$

where $J_{\text{active}} = \mathbf{x}^T(0)\mathbf{P}(0)\mathbf{x}(0)$ and $u_{\text{active}}(t)$ becomes the steady-state optimal solution of an active case computed from Eqs. (6) and (7) with $\dot{\mathbf{P}}(t) = 0$.

Eq. (11) shows that the performance index of a semi-active system is always greater than that of the optimal active system, which depends only on initial conditions. It also reveals that the minimization of the performance index, J_{semi} , is identical to the minimization of the integral term since J_{active} does not depend on $u_{\text{semi}}(t)$. Obviously, the integral term, which is a quadratic in $(u_{\text{semi}}(t) - u_{\text{active}}(t))$, is minimized by the optimal solution $u_{\text{semi}}(t) = u_{\text{active}}(t)$. However, in the application of the semi-active tuned mass damper, the control force $u_{\text{semi}}(t)$ may not be able to produce such optimal force; therefore the solution of a semi-active system is an optimal solution which minimizes the above equation while satisfying its constraint in producing the control force.

Using the calculus of variations and assuming that the semi-active control force $u_{\text{semi}}(t)$ becomes bounded, the optimal solution to Eq. (11) for the deterministic case has been solved by Tseng and Hedrick [10]. The solution is the time-varying solution of the so-called two-point boundary-value problem. Thus the feedback control obtained depends on the future of system dynamics, and consequently it is difficult to implement in actual application. Therefore the class of suboptimal control solution that is more attractive toward the implementation is considered in the following.

Instead of directly minimizing the whole integration, the control law is obtained by minimizing the instan-

taneous error resulting from the difference $(u_{\text{semi}}(t) - u_{\text{active}}(t))$. This leads to the minimization of the function

$$L(t) = (u_{\text{semi}}(t) - u_{\text{active}}(t))^2. \quad (12)$$

Let us consider the STMD having variable damping coefficient, $c(t)$, and the relative velocity, $\dot{z}(t)$, installed on a structure as shown in Fig. 2. By adjusting the damping, $c(t)$, the semi-active control force can be produced as

$$u_{\text{semi}}(t) = -c(t)\dot{z}(t). \quad (13)$$

A situation in which the damping, $c(t)$, can be varied only within a certain range is considered. The variable damping coefficient, $c(t)$, is assumed to be variable in the range

$$c_{\min} \leq c(t) \leq c_{\max}. \quad (14)$$

To find the optimal variation of damping coefficient, which minimizes the performance measure (11) subjected to the inequality constraints (14), we define the Lagrangian [12]

$$H(c(t), \lambda_1, \lambda_2) = \frac{1}{2}L(c(t)) + \lambda_1 g_1(c(t)) + \lambda_2 g_2(c(t)), \quad (15)$$

where

$$g_1(c(t)) = c(t) - c_{\max} \leq 0 \quad (16a)$$

and

$$g_2(c(t)) = c_{\min} - c(t) \leq 0 \quad (16b)$$

are the imposed inequality constraints and $\lambda_1, \lambda_2 \geq 0$ are the Lagrange multipliers.

The necessary condition for a minimum becomes

$$\frac{\partial H}{\partial c} = \frac{1}{2} \frac{\partial L}{\partial c} + \lambda_1 \frac{\partial g_1}{\partial c} + \lambda_2 \frac{\partial g_2}{\partial c} = 0. \quad (17)$$

By substituting Eq. (12) into Eq. (11) and then into Eq. (17), we get

$$c_{\text{opt}}(t) = \frac{\lambda_2 - \lambda_1}{(\dot{z}(t))^2} + \frac{u_{\text{active}}(t)}{\dot{z}(t)} \quad \text{for } \dot{z}(t) \neq 0. \quad (18)$$

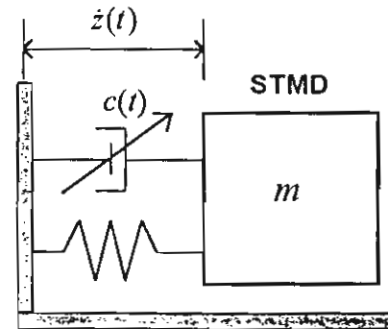


Fig. 2. Semi-active mass damper (STMD) system.

More precisely, we have an optimal variation of damping coefficient as

$$c_{\text{opt}}(t) = \text{SAT} \left(\frac{u_{\text{active}}(t)}{\dot{z}(t)} \right) \quad \text{for } \dot{z}(t) \neq 0, \quad (19)$$

where the SAT function saturates between c_{min} and c_{max} . The sufficient conditions that the obtained solution is minimum are ensured by Eq. (17) and

$$\frac{\partial^2 H}{\partial c^2} = (\dot{z}(t))^2 \geq 0. \quad (20)$$

A graphical interpretation of the value of the optimized parameter, $c_{\text{opt}}(t)$, is plotted in Fig. 3. In this plot, the variation of $c_{\text{opt}}(t)$ occurs only when the term $u_{\text{active}}(t)/\dot{z}(t)$ lies within a variable range. Otherwise it is set to either the maximum or minimum value.

The control law (19) can also be obtained by direct inspection of Eqs. (11)–(13), and it should result in the so-called clipped optimal control [8]. However, the mathematical derivation given here is a more rigorous approach.

3. Vibration control of single-degree-of-freedom (SDOF) system

3.1. Problem formulation and system properties

In the following numerical examples, an STMD is employed to reduce the vibration of the main structure modeled by an SDOF system subjected to an excitation $f(t)$ as shown in Fig. 4. The equations which govern the vibration of the system are

$$(m_1 + m_2)\ddot{x}_1(t) + c_1\dot{x}_1(t) + k_1x_1(t) = f(t) - m_2\ddot{z}(t) \quad (21)$$

and

$$m_2\ddot{z}(t) + c_2(t)\dot{z}(t) + k_2z(t) = -m_2\ddot{x}_1(t), \quad (22)$$

where $x_1(t)$ and $z(t)$ represent, respectively, the displacement of structure and the relative displacement of the

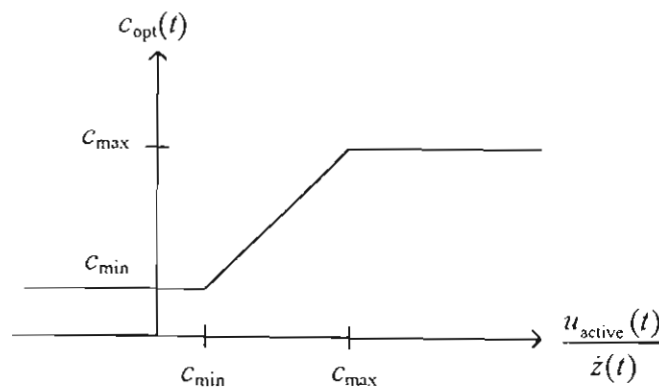


Fig. 3. Optimal value of damping parameter $c_{\text{opt}}(t)$.

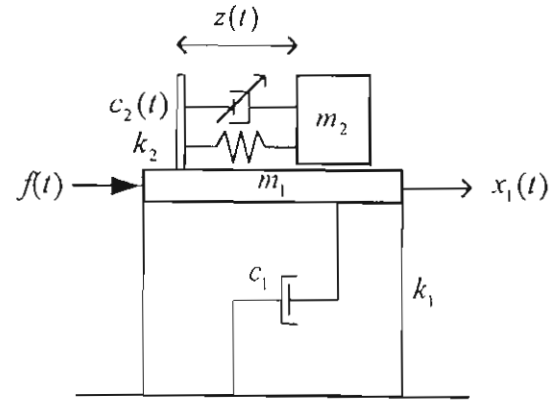


Fig. 4. Structure-STMD model.

mass damper. The time-dependent damping of the STMD, $c_2(t)$, in Eq. (22) is computed according to the control law (19) derived in the previous section.

Throughout the simulations, the main structure is assumed to have the natural frequency of 3.0 Hz and the damping ratio of 2.0%. The STMD is assumed to have the mass ratio of 3.0% and the minimum and maximum damping coefficients of 0.001 and 3.403 N s/m, respectively. In computation of the semi-active control law, the control weight matrix, Q , is assigned such that the term $\mathbf{x}^T(t)Q\mathbf{x}(t)$ in Eq. (2) becomes the vibration energy of the main structure, r is selected to give the corresponding active closed-loop system having two pairs of natural frequencies and damping ratios of $f_1=2.19$ Hz and $\xi_1=21.53\%$, and $f_2=3.68$ Hz and $\xi_2=32.94\%$, respectively. For performance comparison, a TMD with the same mass ratio (3%) is considered and its parameters are optimized according to the type of excitations [13].

3.2. Response under free vibration

The structure is assumed to have the initial velocity of +10 cm/s while the mass damper is assumed to start from rest. Fig. 5 shows the displacement histories of the structure under the TMD and STMD. It can be seen from the figure that the performance found by application of the STMD is significantly better than that of the TMD. In particular, the STMD improves (by a reduction of about 20%) the root-mean-square of the displacement of the structure. However, there is no significant vibration reduction gained from the STMD in the first cycle of vibration. To explain this inefficiency, the power demand and the control characteristic of the STMD are investigated further.

Fig. 6(a) and (b) shows the power demand and the normalized damping variation of the STMD, respectively. In Fig. 6(a), the time history of the semi-active power that is done by the semi-active system and the desired power history that is indeed required by the corresponding active system are compared. These power

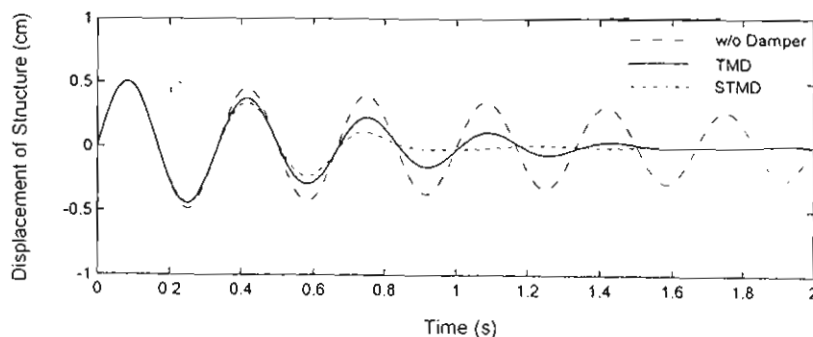


Fig. 5. Free-vibration responses under TMD and STMD.

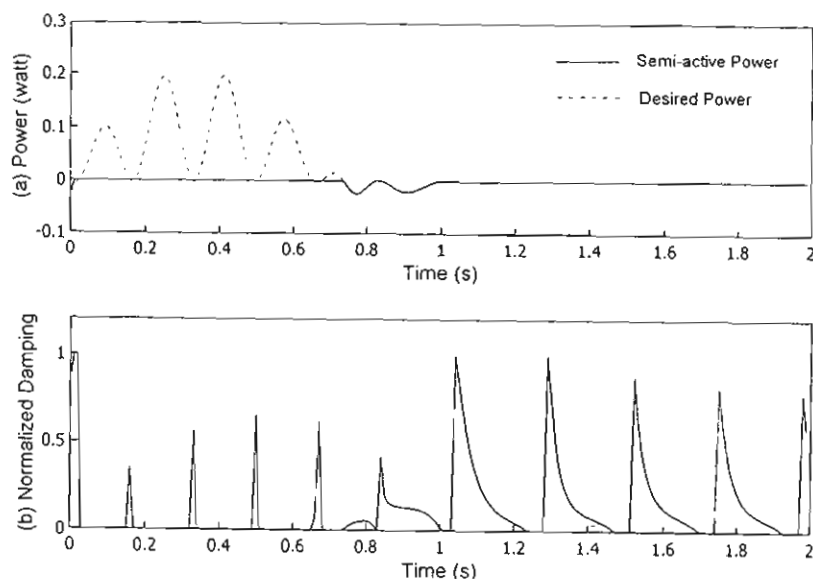


Fig. 6. Free-vibration responses of STMD: (a) power demand and (b) damping variation of STMD.

histories are calculated from $P(t)=u(t)\dot{z}(t)$, where $u(t)$ are set to $u_{\text{semi}}(t)$ for semi-active power and $u_{\text{active}}(t)$ for the desired power. It should be noted that positive power means a supply of energy to the system, and negative power means a dissipation of energy from the system. Clearly, the plot reveals that the STMD requires a supply of energy for the first few cycles. In fact, the mass damper needs sufficient dynamic with proper phase before it can effectively suppress the structure's motion since the magnitude of control force resulting from the mass damper depends on the dynamics of the mass damper itself. However, the STMD, which utilizes the modulation of damping to achieve the performance of an active system, cannot supply the energy into the system because the dashpot always dissipates the energy. Therefore, for the first 0.80 s, the damping of the STMD is set to minimum in order to maximize the energy transfer from the structure. Nevertheless, this control characteristic is not effective in the suppression of transient vibration. A similar incapability due to the lack of sufficient dynamics was also found in the case of using

TMD, as reported by Walsh and Lamancusa [2], Tanaka and Kikushima [14] and Abe and Igusa [15].

3.3. Response under harmonic excitation

The linear structure attached to the STMD, Eqs. (21) and (22), is no longer linear but piecewise linear. It is known that for a non-linear system the response magnification factor may depend on the excitation, or the system response may not be periodic. However, following the proof of Hac and Youn [16], it can be demonstrated that the responses of Eqs. (21) and (22) to periodic excitation are periodic with the same period as the excitation, and the amplitude ratio is independent of the excitation amplitude. Therefore the following comparison of the control performance in the frequency domain is meaningful.

Fig. 7 shows typical time histories of the structure subjected to harmonic excitation under TMD and STMD. Assuming that the system starts from rest, the steady-state amplitude and the peak amplitude are

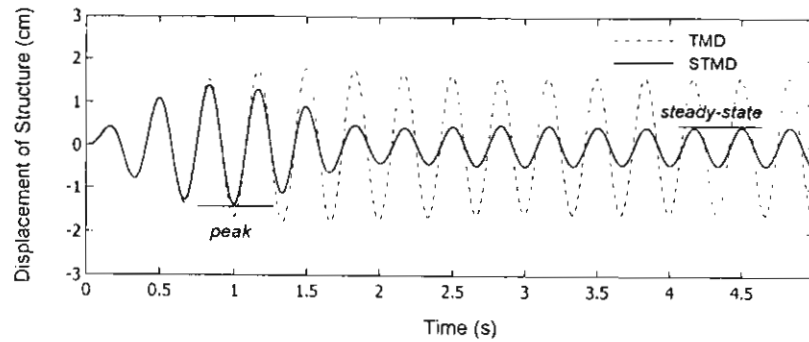


Fig. 7. Typical harmonic responses of the structure under TMD and STMD (excitation frequency of 3.0 Hz).

defined as shown in this figure for STMD. Considering the frequency range of 2.0–4.0 Hz, Fig. 8 plots the steady-state amplitude responses of the structure under TMD and STMD. It indicates that a substantial improvement of steady-state amplitude reduction of the structure, compared with TMD, can be achieved by STMD. Under STMD, the suppression band becomes wider and the steady-state amplitude of the structure inside this band is considerably smaller. In particular, at the resonant frequency ($f_n=3$ Hz), the steady-state response of the structure under STMD becomes 33.1% of that under TMD. These control improvements are greater than those of an increase of the TMD's mass by about four times (see case $\mu=0.12$). However, like TMD, employing STMD creates two peaks of the structure response found at $f=2.5$ and 3.3 Hz. It is noted that, beyond this frequency range (2.5–3.3 Hz), there is no significant difference between the steady-state responses under TMD and STMD. Fig. 9 compares the peak amplitude responses of the structure under TMD and STMD. Since the peak amplitude of the structure always occurs within a few cycles of vibration, the control improvement of STMD is expected to be lower than that in the steady-state amplitude case. This is due to the lack of sufficient amplitude of STMD as previously explained in free-vibration conditions. However, the figure reveals that the

STMD can significantly improve the peak amplitude of the structure over TMD, especially around the natural frequency of the structure, at which a 20% improvement can be observed.

4. Conclusion

A semi-active tuned mass damper (STMD) with variable damping under harmonic excitation is studied. An optimal control law, which minimizes the quadratic performance index for the STMD, has been derived. By minimizing the instantaneous increasing cost from the performance index of the corresponding optimal active system, it is shown that the semi-active optimal control becomes a clipped optimal control law.

The control of a single-degree-of-freedom structure attached to an STMD is considered. Through computer simulation, the control performances of the semi-active mass damper under free and harmonically forced vibrations are determined and are characterized by consideration of the power demand.

It is demonstrated that the STMD requires sufficient amplitude (with proper phase) before it can effectively suppress the vibration. The semi-active mass damper with variable damping, though, provides a flexibility of

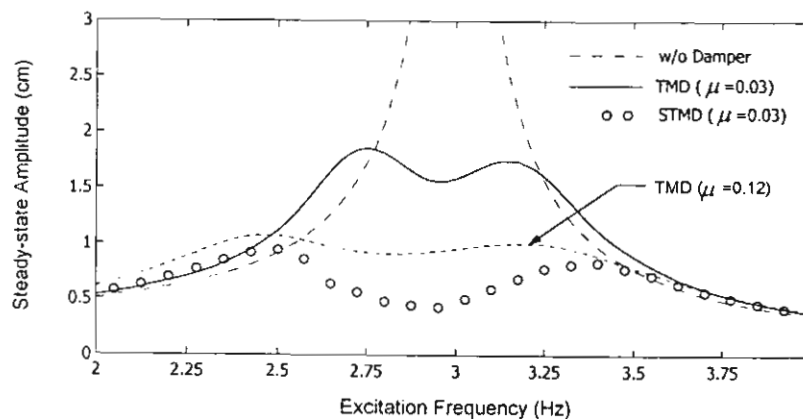


Fig. 8. Steady-state frequency responses of the structure under TMD and STMD.

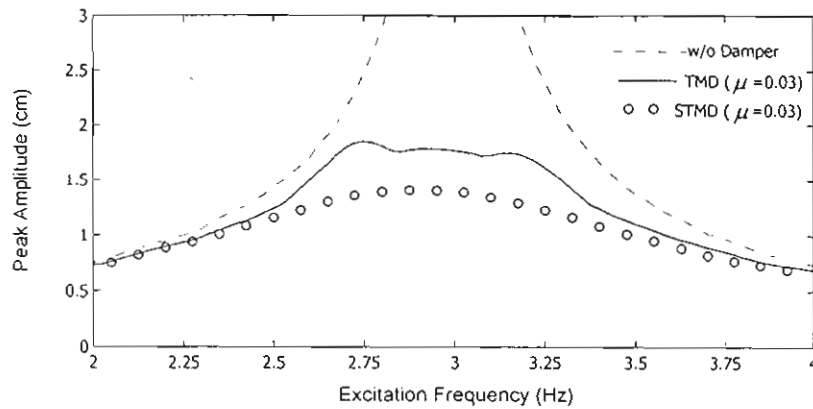


Fig. 9. Peak frequency responses of the structure under TMD and STMD.

varying damping of the mass damper to cope with the variation of disturbance. It has inherently a limitation due to the incapability of introducing energy to the system. Consequently, its control performance is not effective during transient periods. However, it is found that the STMD can substantially improve the steady-state response of the structure around the tuning frequency over the passive TMD. Such an improvement is equivalent to an increase of the TMD's mass by about four times.

Generally, the results obtained in this study indicate that the STMD, which is usually cheaper than the active damper system, offers a substantially better performance than the conventional passive TMD. Consequently, it represents a very attractive alternative for structural control.

Acknowledgements

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Title no. 98-S53

Cyclic Behavior of Reinforced Concrete Structural Walls with Diagonal Web Reinforcement

by Chadchart Sittipunt, Sharon L. Wood, Panitan Lukkunaprasit, and Pichai Pattararattanukul

A series of four reinforced concrete walls were tested to failure to evaluate the influence of diagonal web reinforcement on the hysteretic response. Two walls contained conventional horizontal and vertical web reinforcement, and two walls contained inclined web reinforcement. Reinforcement details were representative of construction practice in regions of low to moderate seismic risk. A single layer of web reinforcement was used, and the transverse reinforcement in the boundary elements did not confine the concrete core.

Both walls with conventional web reinforcement failed due to web crushing. Pinched shapes characterized the hysteresis curves for top displacement and shear distortion near the base. In contrast, the walls with diagonal reinforcement displayed rounded hysteresis curves and failed due to crushing of the boundary elements. The choice of web reinforcement did not have a significant influence on the maximum lateral load resisted by the walls, but measured crack widths were less, and more energy was dissipated by the walls with diagonal reinforcement during loading cycles with comparable levels of displacement. Analytical results from finite element analyses also indicated that the diagonal web reinforcement was effective in transferring shear force to base of walls, especially during load reversals when most diagonal cracks in concrete remained open and compressive struts in concrete were not effective in transferring shear force.

Keywords: reinforced concrete; reinforcement; shear strength; wall.

INTRODUCTION

Observations from previous earthquakes have indicated that well-designed structural walls can be used effectively as the primary lateral-load resisting system for both wind and earthquake loading in multistory buildings.^{1,2} Because it is not economical to design slender structural walls to remain elastic during strong earthquakes,³ the inelastic response of structural walls should be considered during the design process.^{4,5} To survive strong ground motions from severe earthquakes, structural walls must be able to dissipate energy after yielding and should not be susceptible to sudden failures due to shear or local instabilities.⁶

Results from previous investigations^{3,5,7,8} have demonstrated that structural walls that deform primarily in shear and that experience large shear distortions have a lower energy dissipation capacity than structural walls that deform primarily in flexure. In addition, it was found that walls that experience large shear distortions were more likely to fail by web crushing, which is caused by deterioration of the compressive strength of the concrete struts in the web.³ Experimental results^{5,7} have indicated that increasing the amount of conventional vertical and/or horizontal web reinforcement in walls that were susceptible to shear failure did not significantly reduce the inelastic shear distortion nor appreciably improve the energy dissipation capacity. Web crushing fail-

ures were still observed in walls designed with a nominal shear strength that exceeded the nominal flexural strength.⁹

Subsequent analytical studies^{10,11} have indicated that the hysteretic response of walls susceptible to shear failures could be improved if diagonal reinforcement was used in the web. Diagonal web reinforcement provided a more effective mechanism for transferring lateral forces into the foundation, resulted in lower shear strains near the base of the wall, and improved the energy dissipation characteristics.

The measured cyclic response of four slender reinforced concrete structural walls is discussed in this paper. Two walls have conventional web reinforcement and two have diagonal web reinforcement. The experimental parameters studied in this research are the amount and the orientation of the web reinforcement. The experimental results indicate that diagonal web reinforcement can significantly improve the energy dissipation characteristics of the walls and prevent web crushing. The amount of diagonal web reinforcement also has an influence on the magnitude of shear distortion observed in the specimens.

RESEARCH SIGNIFICANCE

Experience during the 1989 Loma Prieta and 1994 Northridge earthquakes has shown that economic losses can be significant in buildings that satisfied the life-safety design criteria in current building codes. As a result, procedures to consider the post-earthquake condition of a building when establishing design limit states are currently being developed. Diagonal web reinforcement in structural walls appears to be one way to control structural damage using conventional methods of construction.

EXPERIMENTAL PROGRAM

Four reinforced concrete wall specimens were constructed and tested at Chulalongkorn University to investigate the influence of diagonal web reinforcement on the hysteretic response of structural walls. The experimental parameters included the amount and orientation of the web reinforcement.

Test specimens

The dimensions of the specimens are shown in Fig. 1. All walls had a barbell-shaped cross section with a web thickness of 100 mm and 250 x 250 mm boundary elements. The overall length of the cross section was 1500 mm. Vertical and diagonal reinforcement was anchored in a 600 mm thick

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base girder that was bolted to the laboratory floor. A 250 mm wide by 500 mm deep beam was cast on top of the wall panel, and a hydraulic actuator was attached to the specimen at mid-depth of the top beam. Lateral loads were applied 2150 mm above the base of the wall.

The primary experimental parameters were the amount and orientation of the web reinforcement. The longitudinal and transverse reinforcement in the boundary elements was the same in all four specimens (Fig. 1(b)). A single layer of web reinforcement was used in all walls. Specimens W1 and W2 were reinforced with conventional horizontal and vertical web reinforcement, while diagonal web reinforcement was used in Specimens W3 and W4 (Fig. 2). Only the web reinforcement is shown in Fig. 2. Longitudinal and transverse reinforcement in the boundary elements and the top beams were the same in all specimens.

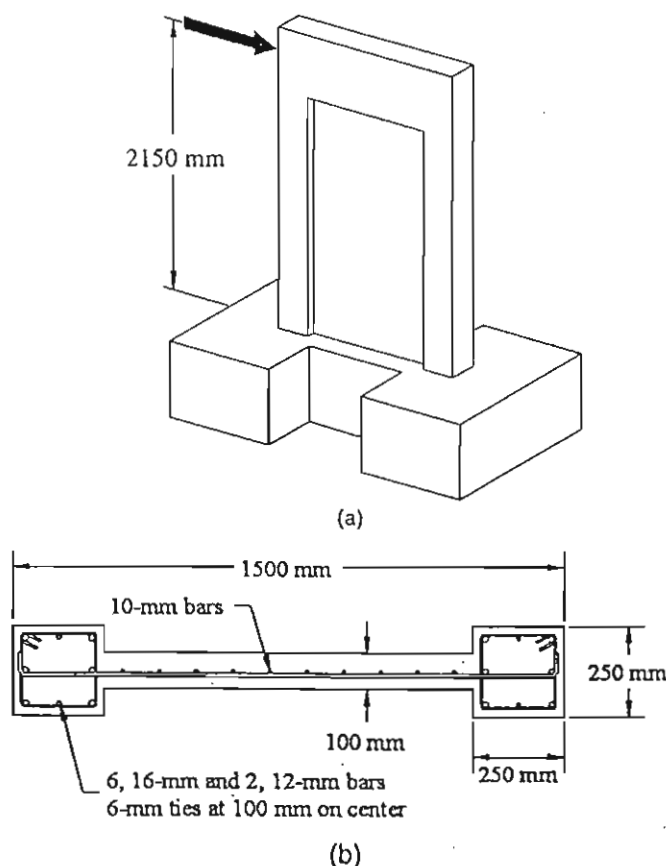


Fig. 1—Dimensions of test specimens: (a) overview of test setup; and (b) cross-sectional dimensions.

The transverse reinforcement in the boundary elements was not intended to provide confinement of the concrete core. The amount of transverse reinforcement provided was consistent with current practice in Thailand and was approximately 1/5 of that required in the U.S. in regions of high seismic risk.

The design procedures in the American Concrete Institute Building Code (ACI 318-99)¹² for regions of low and moderate seismic risk were used to proportion the walls. During design, the concrete compressive strength was assumed to be 29 MPa, and the yield stress of the reinforcement was assumed to be 390 MPa. The measured strengths of the materials used to construct the walls exceeded these values and are summarized in Table 1.

Table 1—Material properties and reinforcement ratios

		W1	W2	W3	W4
Concrete compressive strength, MPa		36.6	35.8	37.8	36.3
Horizontal web reinforcement*	Spacing, mm	150	100	—	—
	Reinforcement ratio, %	0.52	0.79	—	—
Vertical web reinforcement*	Spacing, mm	200	150	—	—
	Reinforcement ratio, %	0.39	0.52	—	—
Diagonal web reinforcement*	Spacing, mm	—	—	150	100
	Reinforcement ratio, %	—	—	0.52	0.79
Longitudinal reinforcement in boundary elements†	Area, mm ²	1430	1430	1430	1430
	Reinforcement ratio, %	2.29	2.29	2.29	2.29
Transverse reinforcement in boundary elements‡	Spacing, mm	100	100	100	100

*Single layer of 10 mm deformed bars, $f_y = 450$ MPa.

†Six 16 mm deformed bars, $f_y = 473$ MPa, and two 12 mm deformed bars, $f_y = 425$ MPa.

‡Single tie, 6 mm plain bars, $f_y = 444$ MPa.

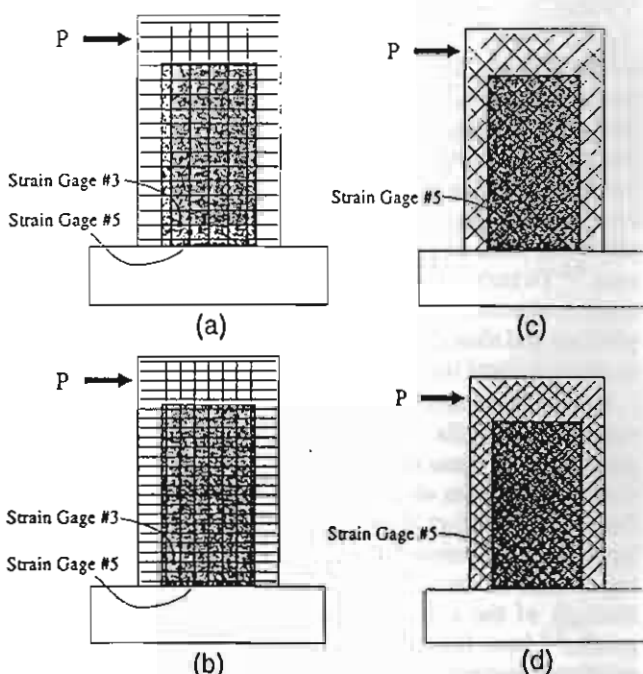


Fig. 2—Arrangement of web reinforcement in test specimens: (a) Specimen W1; (b) Specimen W2; (c) Specimen W3; and (d) Specimen W4.

The reinforcement in Specimen W1 was selected such that the nominal shear and flexural strengths of the wall were nearly equal, based on the assumed material properties. Horizontal web reinforcement was spaced at 150 mm on center, and vertical web reinforcement was spaced at 200 mm on center.

Bar spacings were decreased to 100 mm for the horizontal reinforcement and 150 mm for the vertical reinforcement in Specimen W2. This change in spacing had a negligible influence on the calculated flexural capacity but increased the nominal shear strength by 25% relative to Specimen W1.

The spacing of the web reinforcement in Specimens W3 and W4 was the same as the spacing of the horizontal web reinforcement in Specimens W1 and W2, respectively (Table 1). The web reinforcement in Specimens W3 and W4 was rotated 45 degrees with respect to the longitudinal axis of the wall. The nominal strengths of Specimens W1 and W3 and Specimens W2 and W4 were essentially the same. Nominal capacities calculated using the measured material properties are listed in Table 2.

Test procedure

A photograph of the test setup is shown in Fig. 3. Each specimen was loaded laterally as a vertical cantilever with forces applied through the top beam. No axial load was applied to any of the specimens. The specimens were braced laterally at the top to prevent out-of-plane distortion.

The same nominal loading history was used to test all four specimens described in this paper. Each complete load cycle consisted of one half cycle in the positive direction and one half cycle in the negative direction. The loading history was divided into two parts: load control was used during the first three cycles to push the specimens to the cracking load in

both directions; and displacement control was used in subsequent cycles to push the specimens such that the top deflection reached integer multiples of the yield displacement in both directions. The specimens were pushed to the same displacement level for three complete cycles before the displacement level was increased. The initial yield displacement for each specimen was determined by monitoring strains in the longitudinal steel in the boundary elements. Each test continued until the specimen experienced a significant loss of capacity. Figure 4 shows the loading history of the test program.

Instrumentation

Instrumentation was selected to monitor applied loads, deformations, and strains in the reinforcing steel. Lateral deflections were measured at four levels above the base (0.45, 0.90, 1.50, and 2.10 m). Average shear distortions in the hinging region were calculated from deformations measured along diagonals of a square located in the lower 900 mm of the web. The procedure used to calculate the average shear distortion from the measured data is shown in Fig. 5.

EXPERIMENTAL RESULTS

Table 2 summarizes calculated nominal capacities, maximum loads, and observed failure modes of the specimens. All four specimens sustained maximum loads that exceeded the calculated nominal capacities. The walls with diagonal web reinforcement resisted higher loads than the companion walls with conventional reinforcement; however, the increase in strength was not significant for the walls with higher web reinforcement ratios. Figure 6 shows crack patterns after failure for each of the specimen. Two modes of failure were observed in the specimens: web crushing in Walls W1 and W2; and crushing of the boundary elements in Walls W3 and W4.

Table 2—Calculated and measured capacity of specimens

Specimen	Calculated per ACI 318-99*		Observed response		
	Flexural capacity, kN	Shear capacity, kN	Maximum load, kN	Load at web crushing, kN	Mode of failure
W1	496	482	491	351	Web crushing
W2	515	621	608	350	
W3	518	485	569	—	Crushing of boundary element
W4	545	622	618	—	

*Calculated using measured material properties.

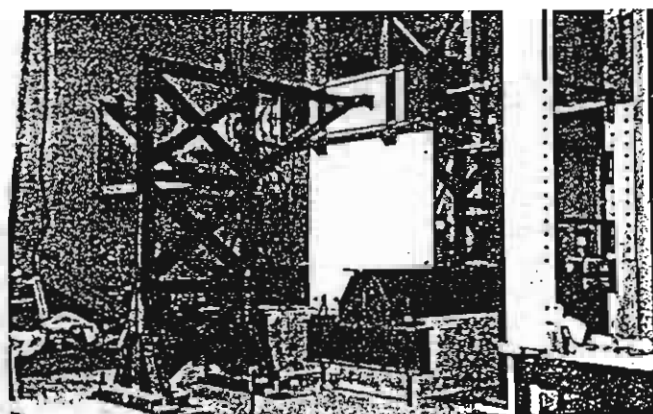


Fig. 3—Photograph of experimental setup.

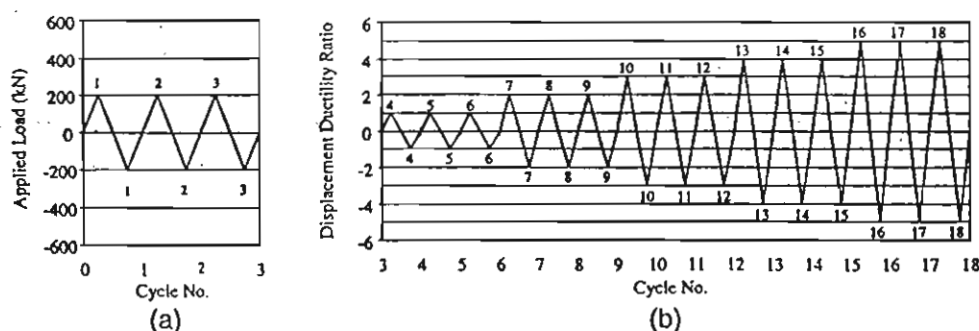


Fig. 4—Loading history for experimental program: (a) load-controlled cycles; and (b) displacement-controlled cycles.

Walls W1 and W2 failed abruptly due to web crushing. Extensive damage to the concrete in the web may be observed in Fig. 6. The applied load at the onset of web crushing was approximately the same for the two specimens. This load was slightly more than 70% of the maximum load resisted during previous loading cycles by Wall W1 and slightly less than 60% of the maximum load resisted by Wall W2. Similar reductions in shear strength with cycling have been observed in previous experimental investigations.¹³

Walls W3 and W4 failed when the concrete in the boundary elements crushed. Although each wall was subjected to a maximum shear force greater than that resisted by its companion wall with conventional reinforcement, the concrete in the web of Walls W3 and W4 was still in good condition after failure. This observation indicates that different shear

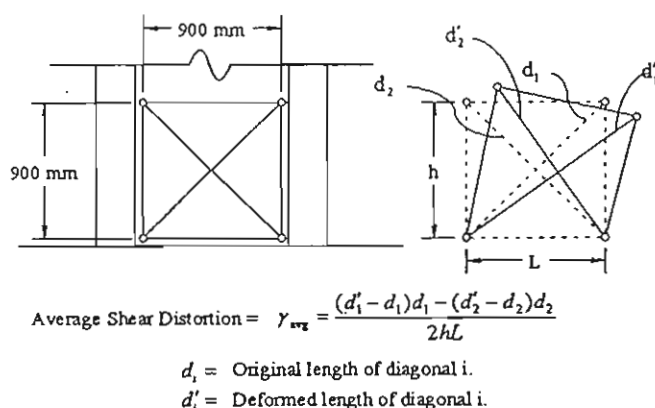


Fig. 5—Estimation of average shear distortion from measured data.⁵

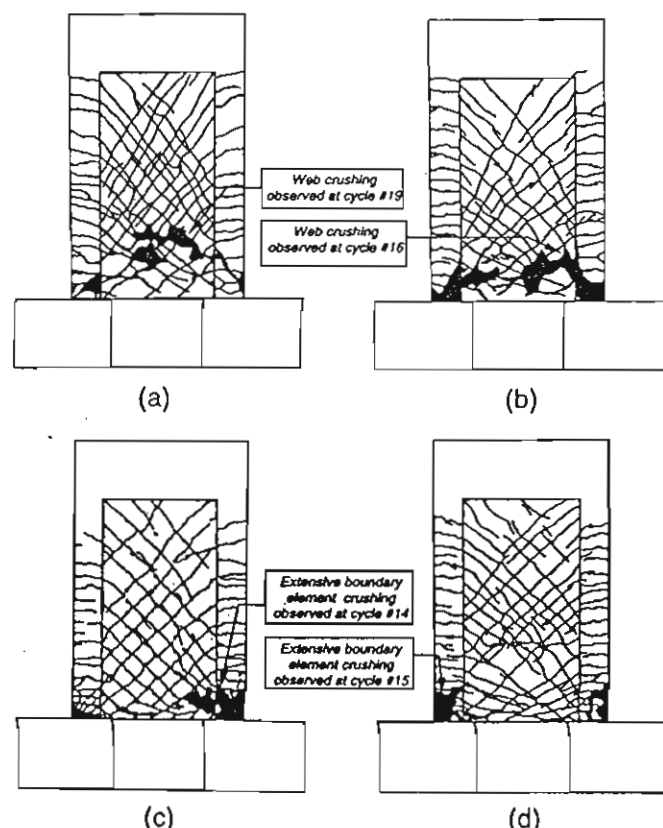


Fig. 6—Observed crack patterns after failure of test specimens: (a) Wall W1; (b) Wall W2; (c) Wall W3; and (d) Wall W4.

transfer mechanisms controlled the behavior of the walls reinforced with conventional web reinforcement and walls reinforced with diagonal web reinforcement. It should also be noted that the boundary element crushing observed in Walls W3 and W4 was not unexpected, given the modest amount of transverse reinforcement in the boundary elements. Significant increases in the displacement capacities of walls were observed in previous tests when the amount of confinement reinforcement in the boundary elements was increased.⁵

Overall hysteretic response

Continuous plots of applied load versus top deflection ($P-\Delta$) for Specimens W1 to W4 are shown in Fig. 7. Numbers shown in each graph correspond to the cycle number during selected cycles in the positive direction. Examination of Fig. 7 clearly indicates the difference between the $P-\Delta$ relationships of the walls with conventional web reinforcement (W1 and W2) and the walls with diagonal web reinforcement (W3 and W4).

The $P-\Delta$ relationships of Walls W1 and W2 experienced significant pinching caused by the low shear stiffness of the walls in hinging regions. In the conventionally reinforced walls, applied shear force is transferred to foundation by compressive struts in the concrete. When the applied load was close to zero, most cracks in the lower portion of the wall remained open, and the compressive struts could not transfer compressive force effectively. This significantly reduced the shear stiffness in the hinging region of the wall and caused large shear distortions at low levels of applied load.

The $P-\Delta$ relationships of Walls W3 and W4 exhibited a rounded shape and did not experience significant pinching. This is due to the fact that part of shear force was transferred to the foundation directly by diagonal reinforcing steel in walls with diagonal web reinforcement. As a result, the shear stiffness in the hinging region was also controlled by the reinforcing steel and did not deteriorate as inelastic shear deformation increased.

Shear distortion at base of walls

Continuous plots of applied load versus shear distortion ($P-\gamma$) in the lower 900 mm of the specimens for Walls W1 to W4 are shown in Fig. 8. The $P-\gamma$ relationships of Walls W1 and W2 also experienced significant pinching. Wall W1 experienced larger shear distortions than Wall W2, especially in the early loading cycles. The $P-\gamma$ relationships of Walls W3 and W4 did not display a pinched shape but were unsymmetrical in later cycles. Shear distortions in the positive direction were greater than those in the negative direction.

Because the amplitudes of the imposed displacements were not the same for corresponding loading cycles for the four walls, normalized parameters were used to compare the hysteretic characteristics of the walls. For each loading cycle, the maximum ductility ratio, defined as the ratio of the maximum displacement during that cycle to the yield displacement, was calculated. The accumulated ductility ratio was then defined as the maximum ductility ratio for a given cycle plus the sum of the maximum ductility ratios in all previous cycles.

Figure 9 shows the relationship between shear distortion and accumulated ductility ratios for all the specimens at half-cycle increments. At a given level of accumulated ductility, the shear distortions experienced by Walls W1 and W2 were nearly equal; however, Wall W3 experienced nearly twice as

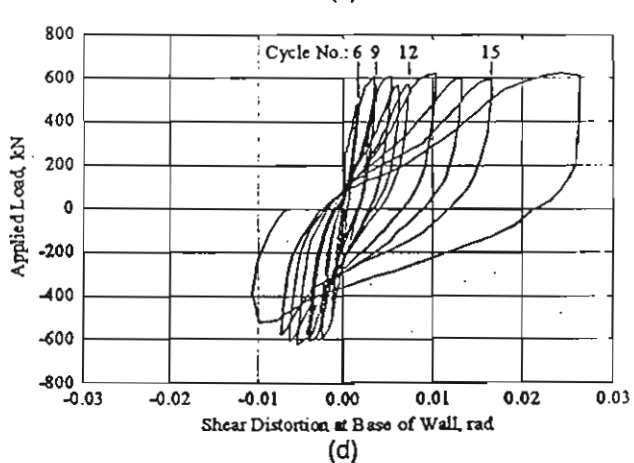
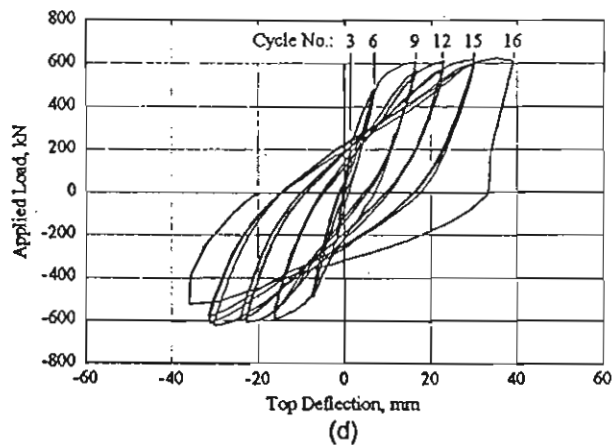
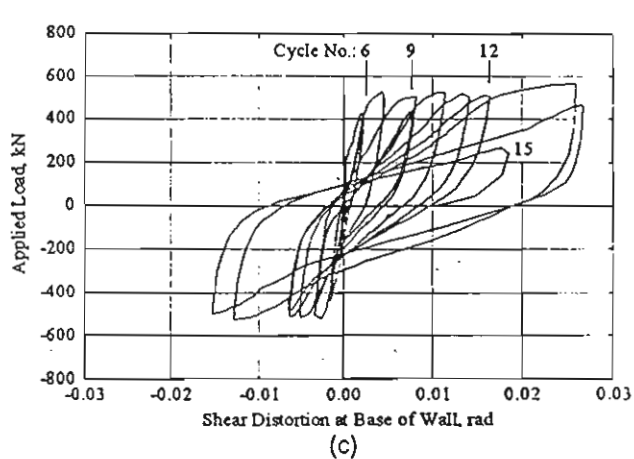
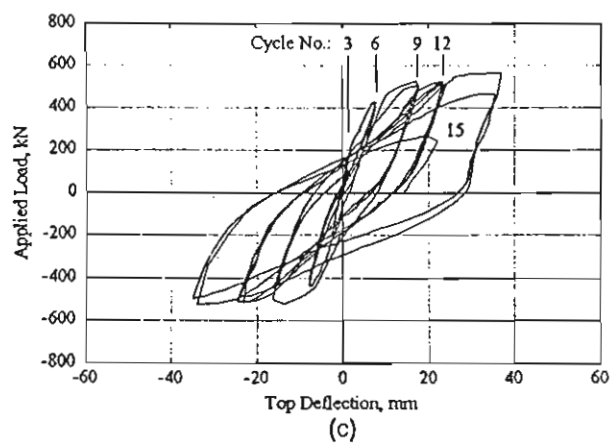
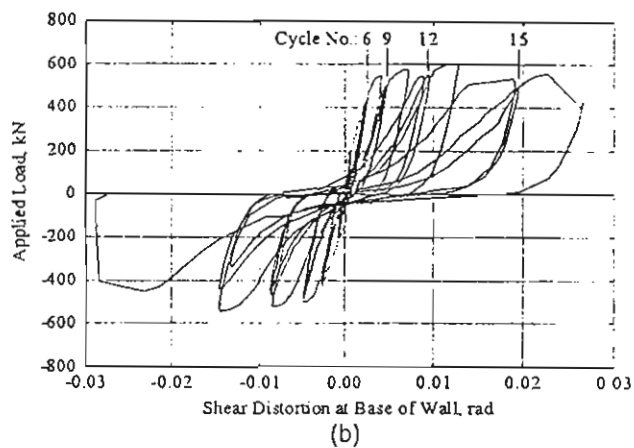
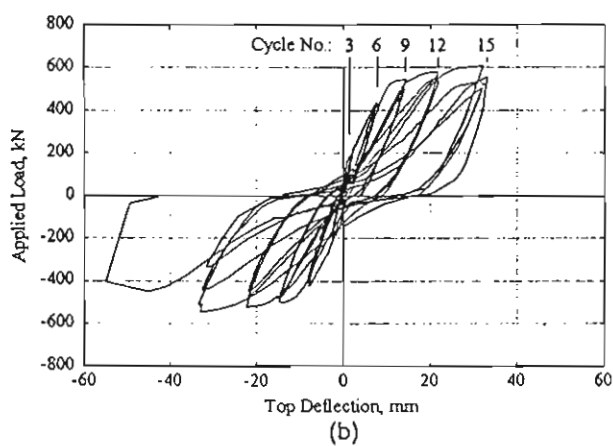
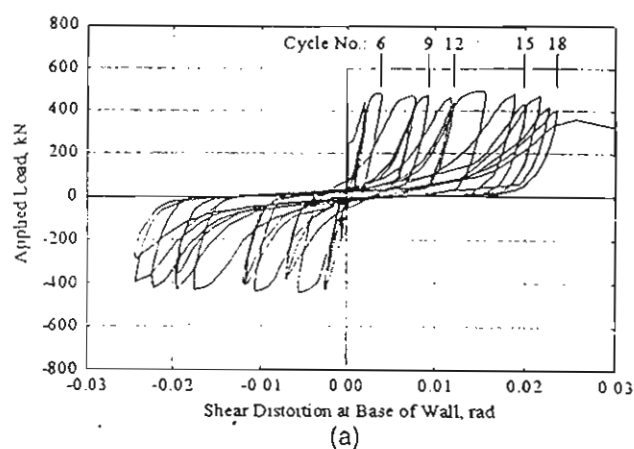
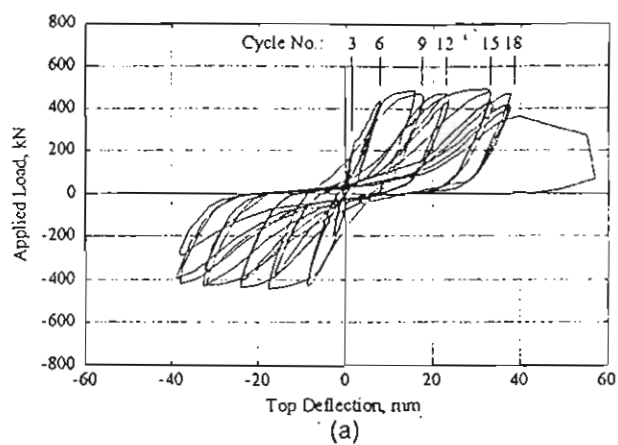


Fig. 7—Overall hysteretic response of walls: (a) Wall W1; (b) Wall W2; (c) Wall W3; and (d) Wall W4.

Fig. 8—Shear distortion at base of walls: (a) Wall W1; (b) Wall W2; (c) Wall W3; and (d) Wall W4.

much shear distortion as Wall W4. This observation shows that increasing the amount of diagonal web reinforcement significantly decreased the shear distortion in the hinging region, while increasing the amount of conventional web reinforcement did not. Nor did increasing the amount of conventional web reinforcement prevent web crushing failures.

Strains in web reinforcement

Differences in the response of the test specimens may also be observed by considering the measured strains in the web reinforcement. The locations of the representative strain gages discussed in this section are shown in Fig. 2.

The largest strains were recorded in the vertical web reinforcement in Walls W1 and W2 (Fig. 10). Yielding was observed during Cycle 7 for the vertical reinforcement in Wall W1 and during Cycle 4 for the vertical reinforcement in Wall W2. Measured strains in the horizontal web reinforcement in Walls W1 and W2 are shown in Fig. 11. Permanent tensile strains were observed in both the vertical and horizontal bars, but the maximum strains in the vertical reinforcement were nearly an order of magnitude larger than those in the horizontal bars.

Measured strains in the diagonal reinforcement in Walls W3 and W4 are shown in Fig. 12. The diagonal reinforcement in both walls yielded in tension during Cycle 7 and remained in tension throughout the remainder of the loading history.

Yielding of the diagonal reinforcement led to an increase in the shear distortions at the base of the walls in later cycles. Figure 13 shows the relationships between shear distortions and strain in the diagonal reinforcement at the maximum and minimum displacement during each loading cycle for Walls W3 and W4. This figure clearly shows that shear distortion in the lower portion of the walls increased significantly after the yielding of diagonal web reinforcement. Before the diagonal reinforcement yielded, the shear distortion near the base of the walls was within the range of -0.002 to 0.002 . After the diagonal reinforcement yielded, the maximum shear distortion was an order of one level of magnitude larger.

Energy dissipation capacity

To survive a severe earthquake, structural walls must be able to dissipate energy. The energy dissipation mechanisms in a given wall depend on the inelastic behavior of the

materials and the orientation of the reinforcement. Walls with conventional web reinforcement transfer shear through compressive struts and aggregate interlock within the concrete and dowel action of the web reinforcement. These mechanisms degrade when subjected to cyclic deformations. In contrast, walls with diagonal web reinforcement transfer shear through tensile forces in the web reinforcement. This energy dissipation mechanism was stable and did not degrade with cycling during the tests. Therefore, walls with diagonal web reinforcement exhibit better energy dissipation characteristics than walls with conventional reinforcement.

The accumulated energy was used in this investigation to compare quantitatively the behavior of the walls. As shown in Fig. 14, the area enclosed by the overall hysteresis curve was calculated for each cycle. The accumulated energy dissipated was then defined as the sum of the area enclosed by all previous hysteresis loops.

The relationship between the accumulated energy dissipated and the accumulated ductility ratio for all four walls is plotted in Fig. 14. Accumulated energy increased nearly linearly with the accumulated ductility ratio for the four walls. The rate of increase was considerably higher for the walls with diagonal web reinforcement, indicating their ability to dissipate more energy at a given level of distortion. This confirms the qualitative observations based on the shape of the hysteresis curves.

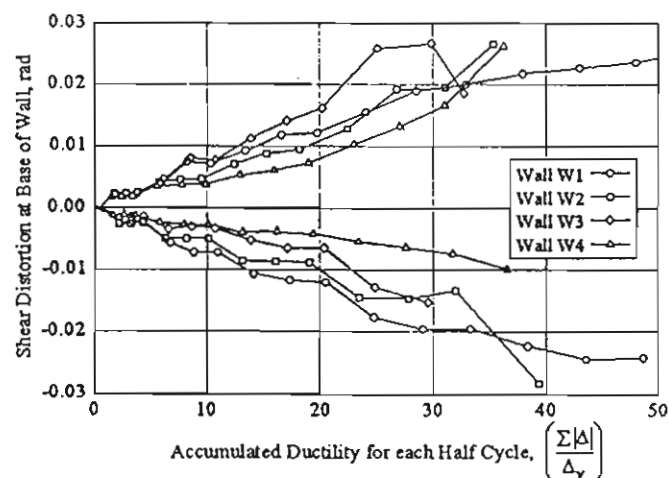


Fig. 9—Variation of shear distortion at base of walls with accumulated ductility.

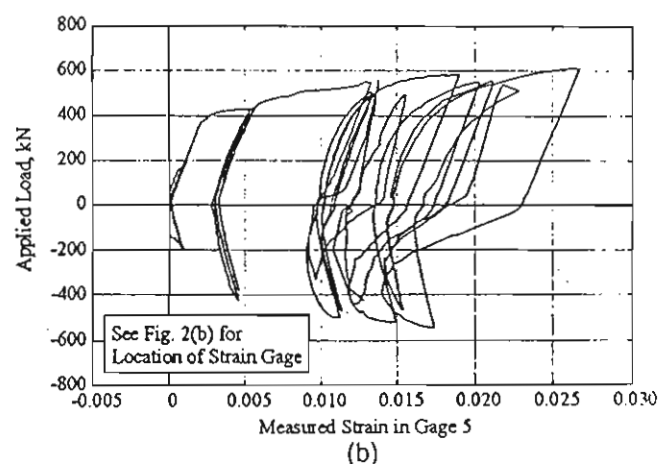
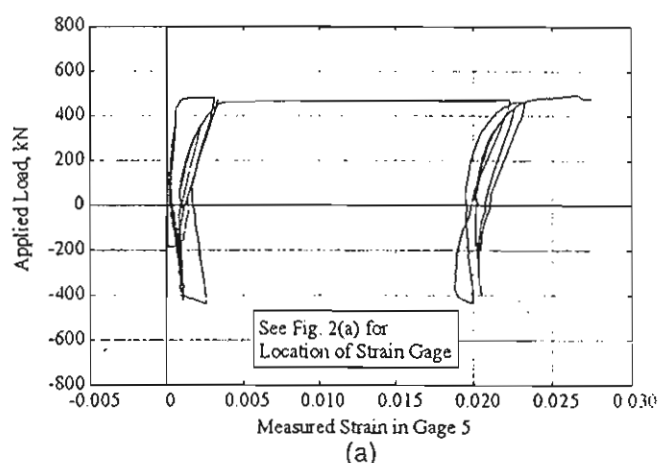


Fig. 10—Measured strains in vertical web reinforcement: (a) Wall W1; and (b) Wall W2.

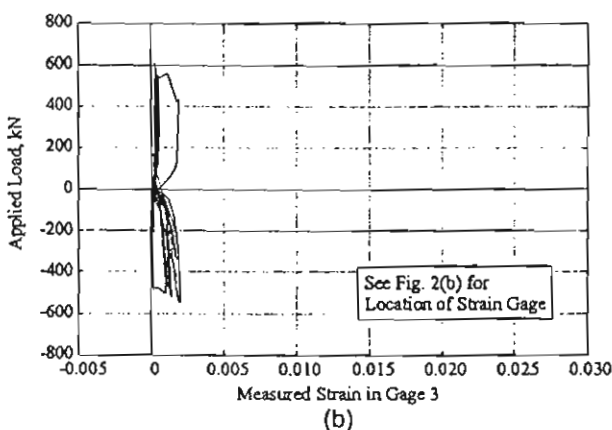
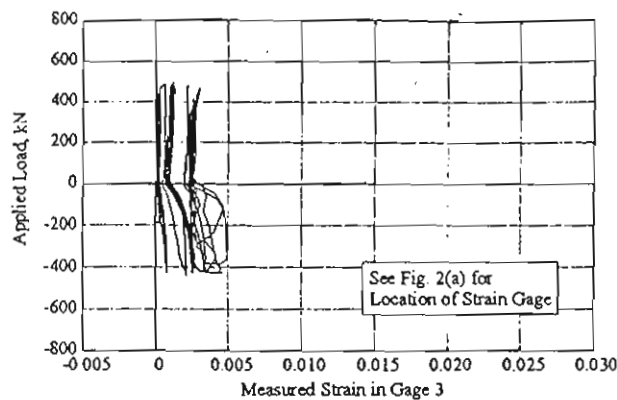


Fig. 11—Measured strains in horizontal web reinforcement: (a) Wall W1; and (b) Wall W2.

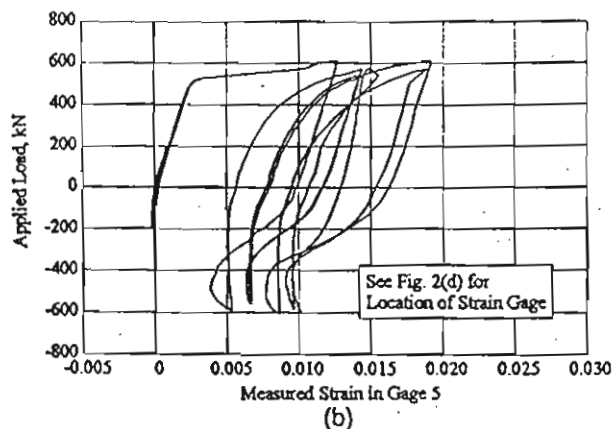
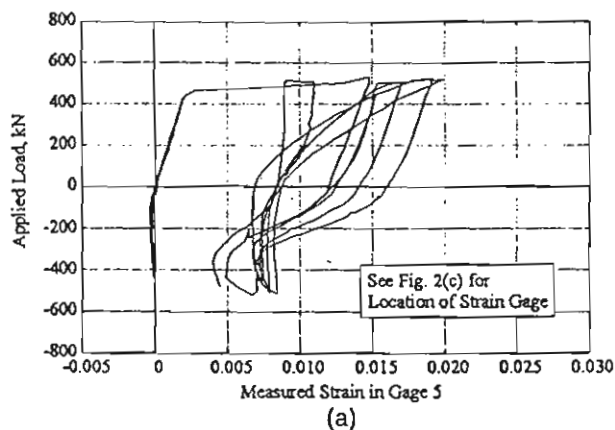


Fig. 12—Measured strains in diagonal web reinforcement: (a) Wall W3; and (b) Wall W4.

ANALYTICAL RESULTS

Figure 15 shows idealized shear transfer mechanisms for walls with conventional web reinforcement and walls with diagonal web reinforcement. Conventional web reinforcement acts as ties that cross diagonal cracks and contribute to

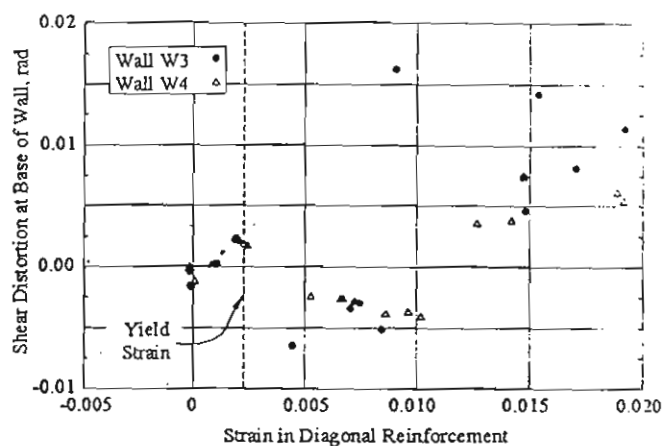


Fig. 13—Variation of shear distortion at base of Walls W3 and W4 with strain in diagonal reinforcement.

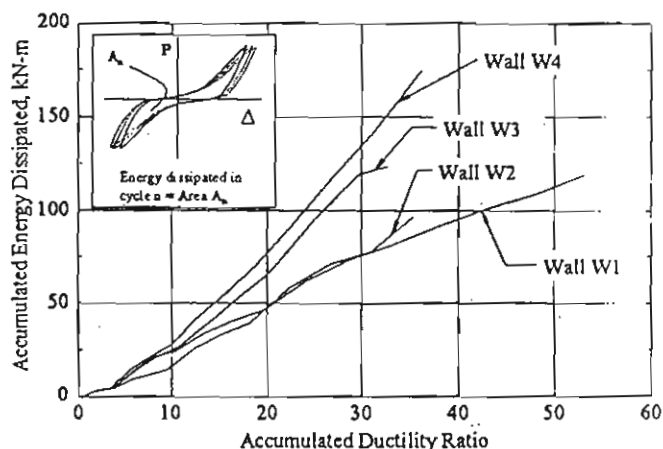


Fig. 14—Variation of accumulated energy with accumulated ductility.

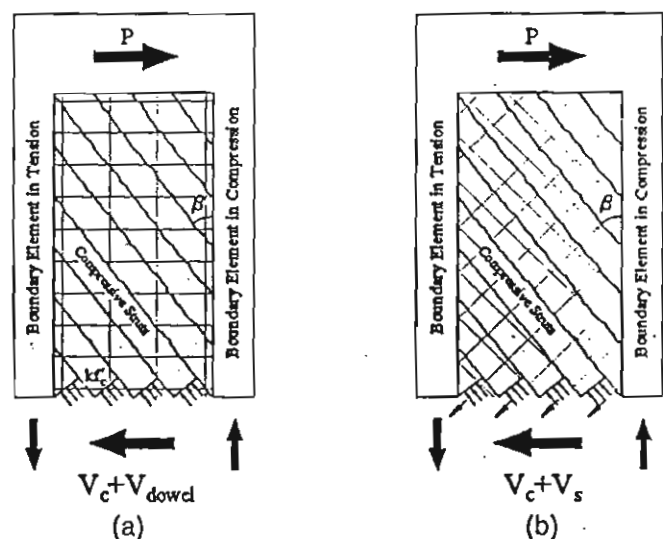


Fig. 15—Mechanisms for transferring shear into foundation: walls with (a) conventional reinforcement; and (b) diagonal reinforcement.

the shear resistance. All lateral force in walls with conventional web reinforcement must be transferred through concrete by compressive struts and aggregate interlock and by dowel action in the reinforcement at the base of the walls ($V_c + V_{dowel}$) (Fig. 15(a)). Web crushing failures occur when the compressive stress exceeds the average compressive strength of the concrete in the strut kf'_c , where k is a factor that represents the decrease in strut compressive strength due to large inelastic shear distortion.³

On the other hand, diagonal web reinforcement helps transfer part of shear force directly to the foundation by tension in the web reinforcement (Fig. 15(b)). As a result, the shear force carried by the compressive struts is reduced. Diagonal shear reinforcement also helps reduce the inelastic shear distortions in the lower portion of walls, thereby reducing the deterioration of compressive strength in concrete struts. The decreases in stress in the concrete struts and in the inelastic shear distortions due to the presence of diagonal web reinforcement improves the shear transfer capacity of concrete in the web and reduces the possibility of web crushing failures.

Finite element analyses were used to investigate shear transfer mechanisms for Walls W3 and W4. The material models used in the analyses are described in previous research.¹⁰ Figure 16 shows the base shear force distribution of Walls W3 and W4 during loading Cycle 12. The total base shear is divided into two components: V_{bs} , the sum of the horizontal components of the forces in all of the diagonal web reinforcement at the base; and V_{bc} , the horizontal component of the shear force carried by concrete at the base.

The results of these analyses indicate that after cycles of inelastic deformation, diagonal web reinforcement is effective in transferring shear force to the foundation, especially at low levels of applied load. The compressive struts are not effective in transferring shear force at low applied loads because most cracks in the web remain open. When the applied load is increased, the shear carried by the concrete also increases because cracks in the web close and compressive struts become more effective in transferring shear force. This explains how diagonal web reinforcement helps reduce pinching behavior and increase the energy dissipation capacity of walls.

It should also be noted that the peak values of the shear in the concrete V_{bc} calculated for both W3 and W4 are nearly the same; however, the maximum value of the shear in the

steel in each wall is controlled by the amount of diagonal reinforcement. The component of shear carried by the steel V_{bs} in both walls is bounded by values of $A_s f_y \cos 45^\circ$ (shown as horizontal broken lines in Fig. 16), where A_s is the total area of diagonal steel in one direction, and f_y is the yield stress of the diagonal reinforcement. This indicates that shear force carried by diagonal web reinforcement at the base of the wall is proportional to the amount of the reinforcement. Although the current ACI design equation provides a conservative estimate of the shear strength of walls with diagonal web reinforcement (Table 2), further study is needed to develop a proper design equation for walls with diagonal web reinforcement.

CONCLUSIONS

This paper summarized the observed cyclic behavior of reinforced concrete structural walls with conventional web reinforcement and with diagonal web reinforcement. The following conclusions are based on the material presented in this paper:

1. Walls with diagonal web reinforcement displayed the ability to dissipate more energy at a given level of lateral deformation than walls with conventional web reinforcement. Increasing the amount of diagonal web reinforcement increased the energy dissipation capacity of the walls, whereas increasing the amount of conventional web reinforcement did not change the energy dissipation capacity of the walls significantly;

2. Walls with diagonal web reinforcement experienced less shear distortion in the hinging region than walls with conventional web reinforcement. Shear distortions did increase, however, after the diagonal web reinforcement yielded. Web crushing was not observed in walls with diagonal web reinforcement. Analytical results indicated that part of the shear force was transferred directly to the foundation by the diagonal reinforcement. Diagonal web reinforcement also helped reduce inelastic shear distortion in the lower portion of walls and, hence, prevent deterioration of the concrete strength in the compressive struts. The decrease in force in the concrete struts and the decrease in inelastic shear distortion improves the shear transfer capacity of concrete in the web and prevents web crushing;

3. Increasing the amount of conventional web reinforcement did not significantly reduce the shear distortion in the hinging regions, nor did it change the observed failure mech-

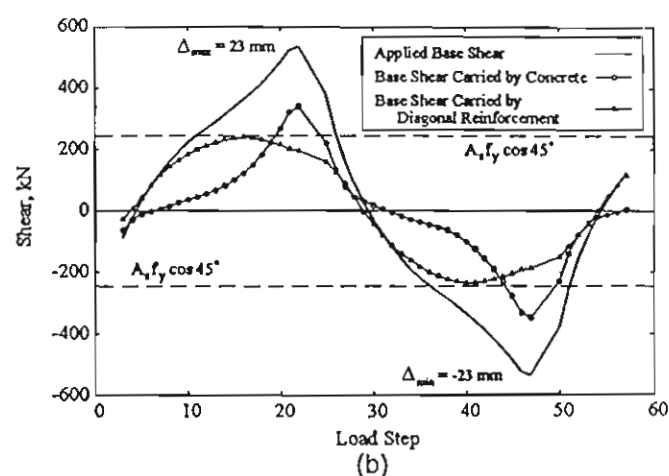
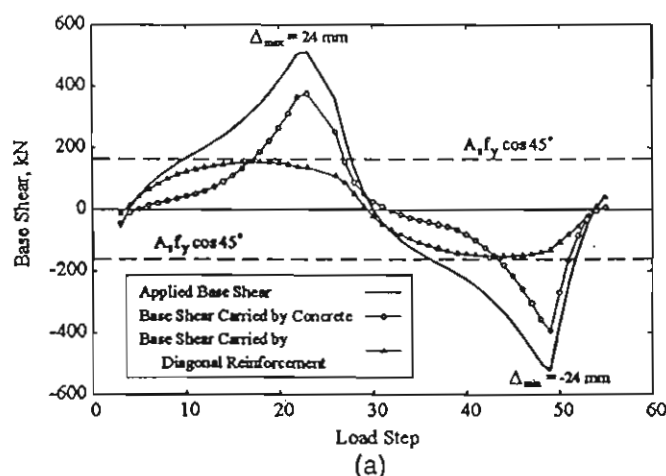


Fig. 16—Calculated distribution of base shear forces: (a) response of Wall W3 during load Cycle 12; and (b) response of Wall W4 during load Cycle 12.

anism. Web crushing controlled the response of both walls with conventional web reinforcement. In walls with conventional web reinforcement, lateral force was transferred to the foundation by compression in concrete struts and dowel action in the reinforcement. Compressive strength of concrete struts deteriorated significantly when the walls were subjected to large inelastic shear distortions. Large shear forces in the compressive struts and deterioration of compressive strength led to crushing of the concrete in the web; and

4. As engineers adopt performance-based design philosophies, new techniques must be developed to control structural damage reliably during earthquakes. Diagonal web reinforcement is one such approach. For loading cycles to a specified lateral displacement, walls with diagonal web reinforcement exhibited smaller crack widths and dissipated more energy than conventionally reinforced walls. In addition, with appropriate confinement of the boundary elements, brittle modes of failure can be avoided. These advantages in performance offset the difficulties associated with placement of diagonal bars during construction.

ACKNOWLEDGMENTS

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TECHNICAL NOTE

Shear modulus and damping ratio of a clay during undrained cyclic loading

S. TEACHAVORASINSKUN*, P. THONGCHIM* and P. LUKKUNAPRASIT*

KEYWORDS: clays; laboratory tests; repeated loading; stiffness.

INTRODUCTION

Research on the cyclic behaviour of clays has mostly concentrated on exploring the undrained cyclic shear strength. The results have led to the establishment of important links between cyclic shear strength and several physical parameters such as stress amplitude, number of load cycles, strain levels, and rate of loading (Wood, 1982). In most of the studies, the undrained cyclic load was continuously applied in either a load-controlled or displacement-controlled manner, called herein the CC test (Fig. 1(a)).

To measure the deformation parameters (i.e. shear modulus and damping ratio) using the cyclic loading test, a staged cyclic loading test (SC test, see Fig. 1(b)) has been widely adopted (Vucetic & Dobry, 1991). After completion of each step, the excess pore water pressure may (or may not) be dissipated. In the present study, dissipation of the excess pore water was allowed, and this caused the initial mean effective stress at the beginning of every load step to be the same. Superimposing of the test results makes it possible to draw the curves of strain level dependence on the shear modulus and damping ratio.

Owing to continuous shaking in actual vibration problems, coupling between the effective stress change and strain level should be considered in determining the shear modulus and damping ratio. This has generally been dealt with by performing a few SC tests at different initial effective stresses or by not allowing dissipation of excess pore water pressure at the end of each step of the test. There is still no information on how the testing procedure affects the shear modulus and damping ratio of clay. The present study aims to present a direct comparison of the test results from the above mentioned two types of cyclic loading tests.

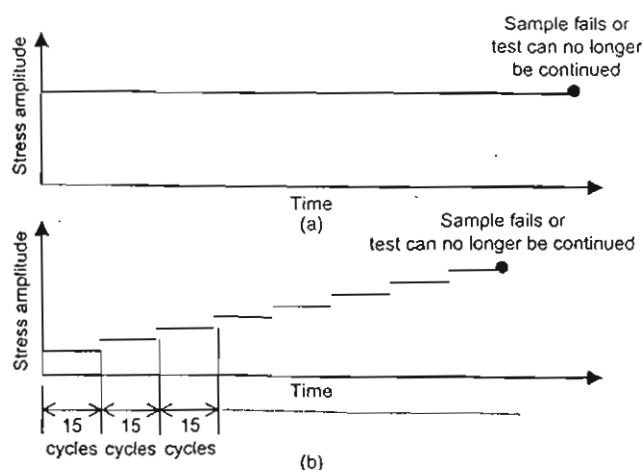


Fig. 1. Testing procedure for: (a) CC test; (b) SC test

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TEST RESULTS

Tests were performed on undisturbed samples collected at a depth of about 6 m from a site in Chulalongkorn University, Bangkok, Thailand. The fundamental properties of soft to medium Bangkok clay have been well addressed by Bergado *et al.* (1990). The test scheme and variation of test conditions are summarised in Table 1. Fig. 2 shows the degradation of the shear modulus with strain during SC and CC tests of the samples having initial isotropic consolidation pressures of (a) 50 and (b) 100 kPa. The shear modulus, G , is computed from the measured Young's modulus, E (using the definition shown at the bottom of Fig. 2(a)), by assuming Poisson's ratio, μ , to be 0.5. Note that the points plotted for CC tests represent only the shear modulus from some selected cycles. The initial shear moduli at strain levels of about 0.02–0.05% were measured for all samples tested. The values at those strains from both groups of tests are in good agreement. This tends to confirm that the quality of the samples is equivalent.

In CC tests, the mean effective stress changed (usually reduced) owing to a change of pore water pressure. The values of the mean effective stress, σ'_f , and the number of cycles, N_f , at the end of each CC test are shown in the figures. The degradation curves obtained from the CC and SC tests are slightly different. Samples tested with the CC procedure exhibit slightly faster reduction in shear modulus with strain. Nevertheless, when the curves are viewed for the whole strain range, it may be reasonable to say that the difference is minor. The shear modulus is dominated much more by strains than by the effective stress changes. Therefore, in the case where there is no information on effective stress change, the curves from SC tests (with dissipation of pore water pressure allowed) could reasonably be adopted to represent the general degradation characteristic. This is in good accordance with the results of cyclic loading tests on Cloverdale clay reported by Zergoun & Vaid (1994).

Figure 3 shows the plot between the damping ratio and strain for the same tests. For the first few cycles of the CC tests, samples show a rather small increase in damping ratio with strain. The curves obtained from CC tests finally merge with that obtained from SC tests. In general, it may be concluded that the damping property obtained from SC tests is close to that of similar samples with similar initial consolidation pressures in CC tests.

Results of tests with different load frequencies (0.1 and

Table 1. Test scheme used in the present study

$(\sigma'_c)_{\text{initial}}$ * kPa	Load frequency: Hz	Type of test	Stress amplitude† (q/p') _{initial}
50	0.1	SC†	Staged
	0.1, 1.0	CC	0.12, 0.20
	0.1, 1.0	CC	0.20, 0.30
	0.1, 1.0	CC	0.30, 0.43
100	0.1	SC†	Staged
	0.1, 1.0	CC	0.16, 0.17
	0.1, 1.0	CC	0.28, 0.26
	0.1, 1.0	CC	0.40, 0.37

* Initial confining stress

† SC test with pore water pressure dissipation

‡ Ratio of peak-to-peak deviator stress ($q = \sigma_1 - \sigma_3$) to initial mean effective stress (p')

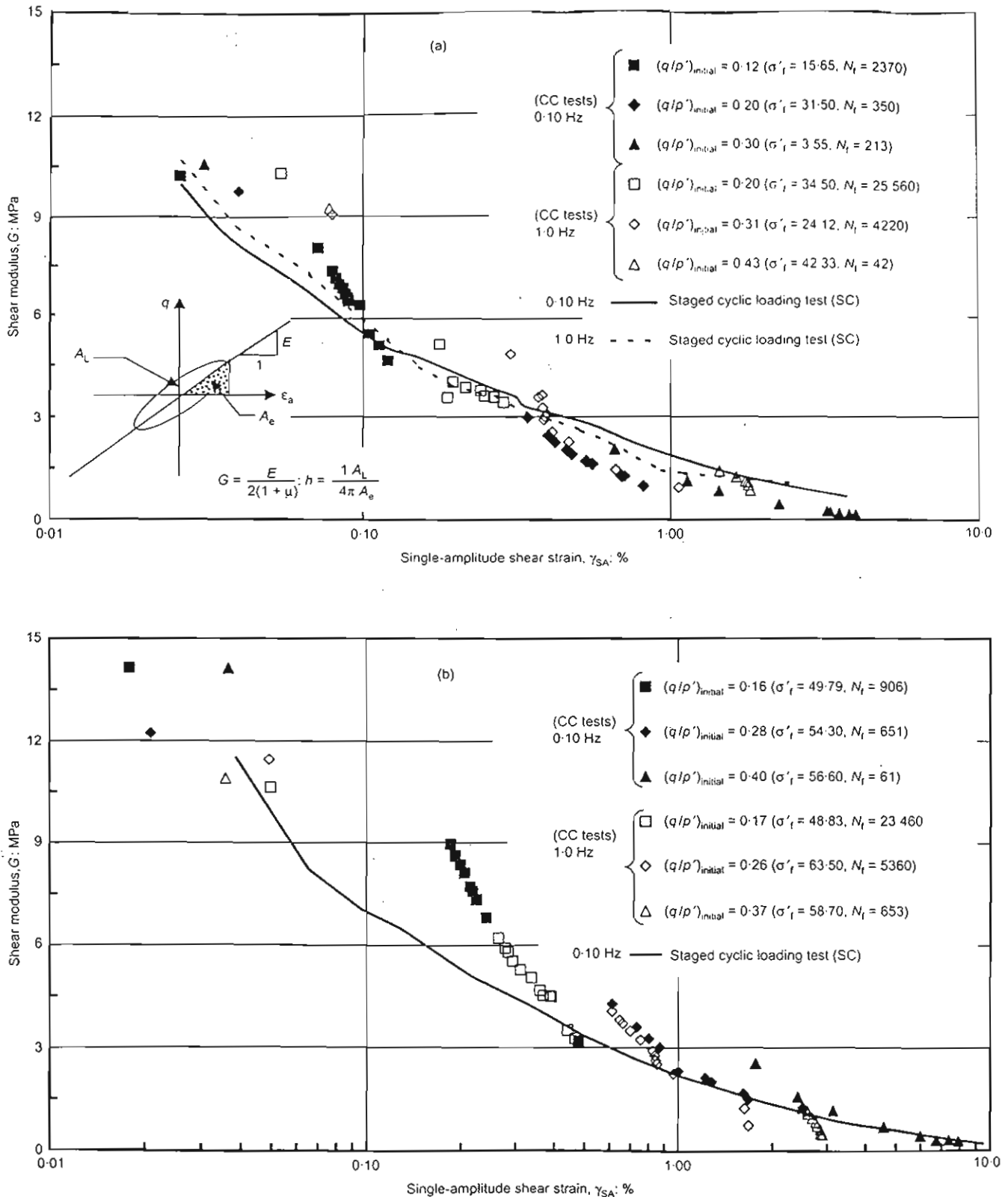


Fig. 2. Shear modulus degradation curve from CC and SC tests: (a) $(\sigma'_c)_{initial} = 50$ kPa; (b) $(\sigma'_c)_{initial} = 100$ kPa

1.0 Hz) are included in Figs 2 and 3 for the purpose of comparison. There is no discernible effect on the shear modulus when the load frequency increases from 0.1 to 1.0 Hz. The damping property is somewhat affected by the load frequency, especially for the results obtained from SC tests (Fig. 3(a)). When load frequency increases from 0.1 to 1.0 Hz, a decrease in the overall damping curve can be observed.

CONCLUSIONS

Cyclic loading test results from two testing procedures (the staged cyclic loading test, SC, and the continuous cyclic loading test, CC) were found to give fairly similar strain dependence characteristics of shear modulus and damping ratio. For a given initial consolidation pressure, strain was a more predominant factor than the effective stress change. The strain dependence

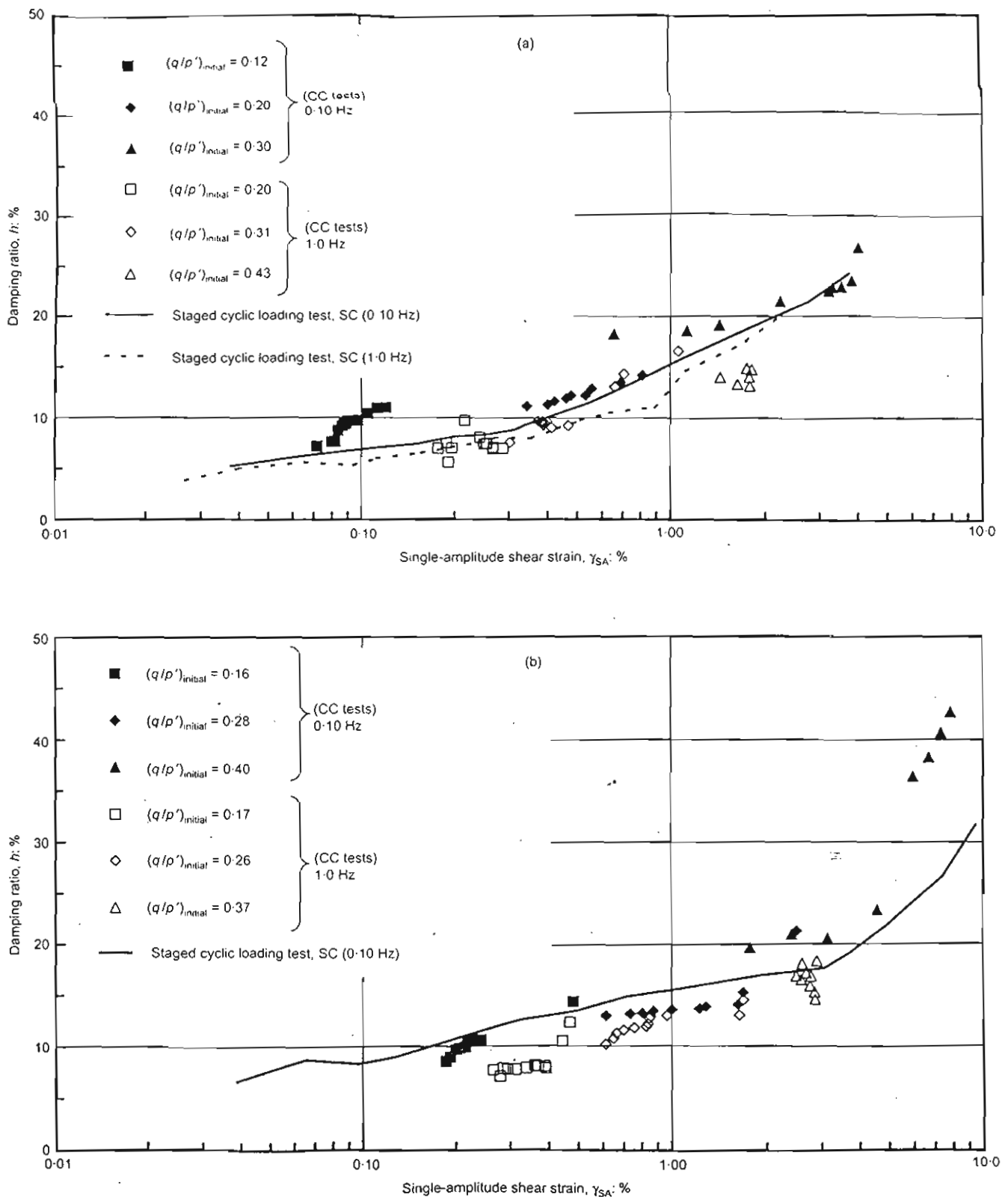


Fig. 3. Damping properties from CC and SC tests (a) $(\sigma'_c)_{initial} = 50$ kPa; (b) $(\sigma'_c)_{initial} = 100$ kPa

curves of shear modulus and damping property drawn from the result of SC tests could probably be accurate enough for practical use.

ACKNOWLEDGMENT

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sustainable development' supported by a grant from the Thailand Research Fund (TRF).

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TECHNICAL NOTE

Stress rate effect on the stiffness of a soft clay from cyclic, compression and extension triaxial tests

S. TEACHAVORASINSKUN*, P. THONGCHIM* and P. LUKKUNAPRASIT

KEYWORDS: clays; dynamics; laboratory tests; shear strength; stiffness.

INTRODUCTION

The rate of applied stress or strain has been one of the most important time effects known to affect the behaviour of soils. In the present study, particular attention was paid to examining the effect of rate of loading on the secant Young's modulus of undisturbed samples of soft Bangkok clay. This was done by direct comparison of the results obtained from undrained triaxial tests carried out under three different loading conditions: compression, extension and cyclic loading.

The effect of rate of loading on the stiffness of clays has been generally studied based on results obtained from the resonant column test and the torsional shear or triaxial test (Stokoe *et al.*, 1994). Though an increase of shear modulus with rate of loading has been reported when the resonant column test is adopted for comparison, the effects of number of loading cycles and stress rate cannot be clearly distinguished. This was partly supported by Shibuya *et al.* (1995). They found that the effect of loading rate on the shear modulus obtained from the cyclic torsional shear test was almost negligible. However, the shear modulus quoted in their study was the equivalent shear modulus, which did not provide a view into the effect of stress reversal.

MATERIAL AND TESTING PROGRAMME

Tests were performed on undisturbed samples collected at a depth of about 6 m from a site in Chulalongkorn University,

Bangkok, Thailand. The natural water content, plasticity index (PI) and maximum past stress of the tested samples varied in the ranges 65–70%, 39–40% and 90–100 kPa respectively. The fundamental properties of soft to medium Bangkok clay have been addressed in detail by Bergado *et al.* (1990) and Sambhandharaksa & Taesiri (1987). The tested samples were first saturated to have a similar degree of saturation ($B \approx 94$ –97%) and then consolidated to the prescribed initial isotropic consolidation pressures: $(\sigma'_c)_{ini} = 50$ and 100 kPa. The initial conditions of the tests conducted are summarised in Table 1. Note that the samples tested with an initial consolidation stress of 50 kPa were lightly overconsolidated.

Shearing under the undrained condition was performed in a stress-controlled manner with three loading types: triaxial compression, triaxial extension and cyclic triaxial tests. The rate of loading, defined in terms of the vertical stress rate, $\dot{\sigma}_v$, ranged from 0.05 kPa/min in a slow monotonic loading test to about 1000 kPa/min in a fast cyclic loading test.

Since the pore water pressure was measured at the top and bottom of the sample with allowed side drainage, discussion regarding the effect of stress rate on the effective stress path may be uncertain, especially in a fast loading test. It is therefore omitted in this paper. Nevertheless, as will be shown later, the effect of stress rate on the secant Young's modulus can be reasonably concluded.

TEST RESULTS

Comparisons between the secant Young's modulus, E , obtained from the staged cyclic and monotonic loading tests are shown in Fig. 1(a) and Fig. 1(b). Before discussion, the follow-

Table 1. Summary of test conditions and some test results

Test no.	Type of loading	Rate of loading* (kPa/min.)	$(\sigma'_c)_{ini}$ † (kPa)	B-value (%)
COM-1	Triaxial Compression	0.5	50	94
COM-2	Triaxial Compression	5.0	50	94
COM-3	Triaxial Compression	50	50	95
COM-4	Triaxial Compression	0.05	100	96
COM-5	Triaxial Compression	0.5	100	96
COM-6	Triaxial Compression	5.0	100	95
COM-7	Triaxial Compression	50	100	94
EXT-1	Triaxial Extension	0.05	50	97
EXT-2	Triaxial Extension	0.5	50	94
EXT-3	Triaxial Extension	0.05	100	96
EXT-4	Triaxial Extension	0.5	100	95
CYC-1	Cyclic triaxial	0.00025‡	100	95
CYC-2	Cyclic Triaxial	0.1‡	100	94
CYC-3	Cyclic Triaxial	0.1‡	50	96

* Defined in term of rate of applied deviator stress, $q = (\sigma_1 - \sigma_3)$.

† Initial consolidation pressure.

‡ Unit in Hz.

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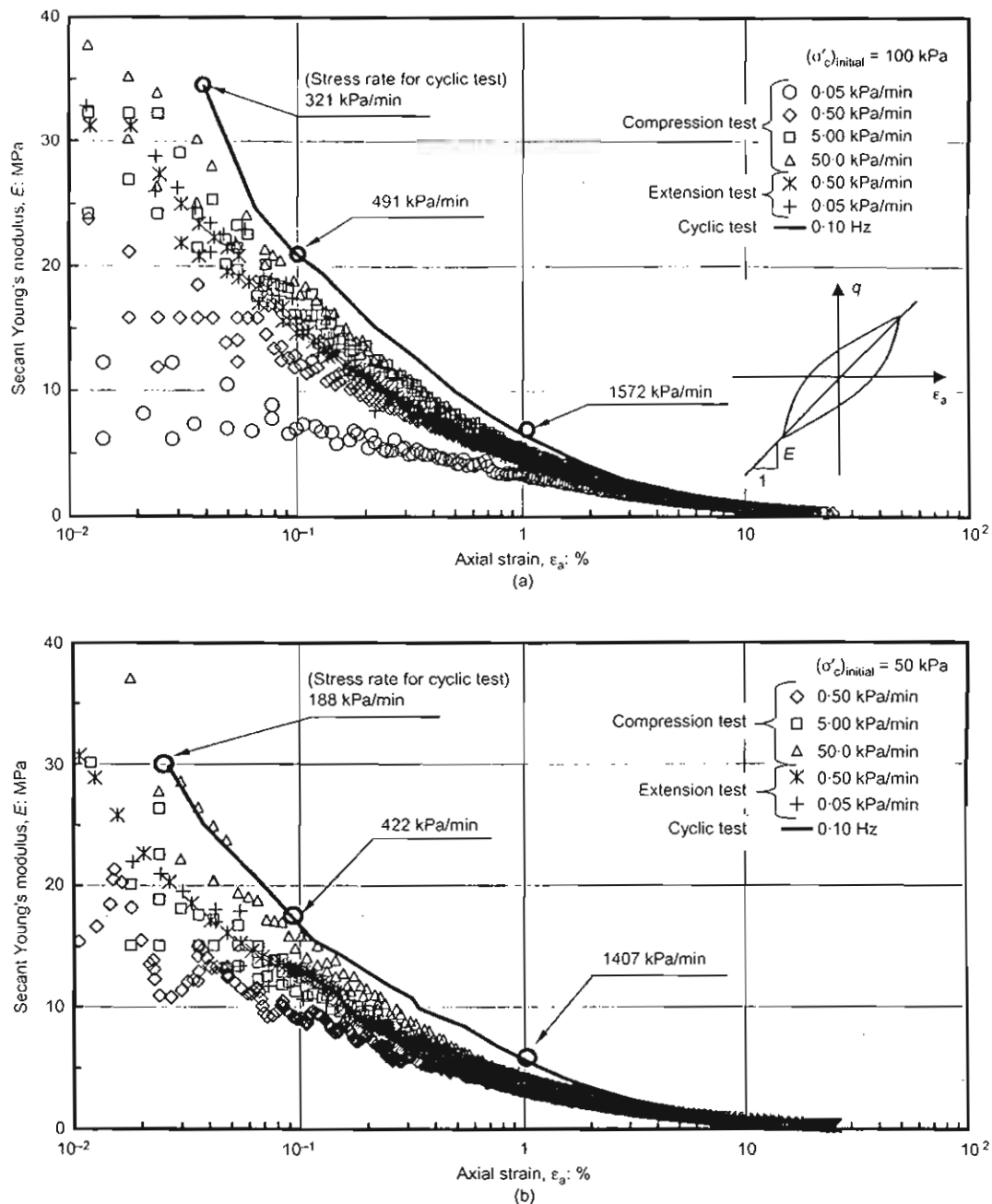


Fig. 1. Variation of secant Young's modulus: (a) $(\sigma'_c)_{\text{initial}} = 100 \text{ kPa}$; (b) $(\sigma'_c)_{\text{initial}} = 50 \text{ kPa}$

ing remarks should be noted on the results of the staged cyclic test (represented by the solid lines in the figures):

- A staged cyclic load test was used with the following procedure. First, 15 cycles of small stress amplitude were applied. After the built-up excess pore water pressure had dissipated, another 15 cycles of loads with a larger stress amplitude were applied. Stress amplitude was increased by 25 kPa per step. This procedure was repeated until the sample failed.
- The secant Young's modulus, E , for a cyclic loading test was defined as shown in Fig. 1(a).
- Since the load frequency was kept constant, the rate of loading increased as the stress amplitude increased, from about 180 kPa/min at moderate strains (at the beginning of the test) to more than 1000 kPa/min at large strains.

The rate of loading affects the values of the measured secant Young's modulus, E . In general, samples tested at a faster loading rate exhibit higher values of E , especially at moderate

strains ($\epsilon_a \approx 0.02$ – 0.2%). For example, from the test results shown in Fig. 1(a), E at an ϵ_a of 0.02% varies from about 10 MPa (in the compression test, stress rate = 0.05 kPa/min) to about 35 MPa (in the cyclic loading test, stress rate = 320 kPa/min). Fig. 2 shows the changes in pore water pressure measured at the top and bottom of the samples during triaxial compressions. Although the pore water pressure measured was not representative for every sample, Fig. 2 implies that the effective stress changes (or the built-up excess pore water pressure) at $\epsilon_a \approx 0.02$ – 0.2% could still be very small. The differences in E at these strains are therefore caused mainly by the difference in rates of loading.

At large strains ($\epsilon_a > 1\%$) the range of loading rates used in the compression tests (0.05 – 50 kPa/min) is too small to initiate any discernible difference in E . Only very high stress rates such as those used in the cyclic loading test ($> 1570 \text{ kPa/min}$) were able to result in a small difference. It may be reasonable to conclude that at large strains the strain level dependence characteristic of E has overcome the effects of loading rate and

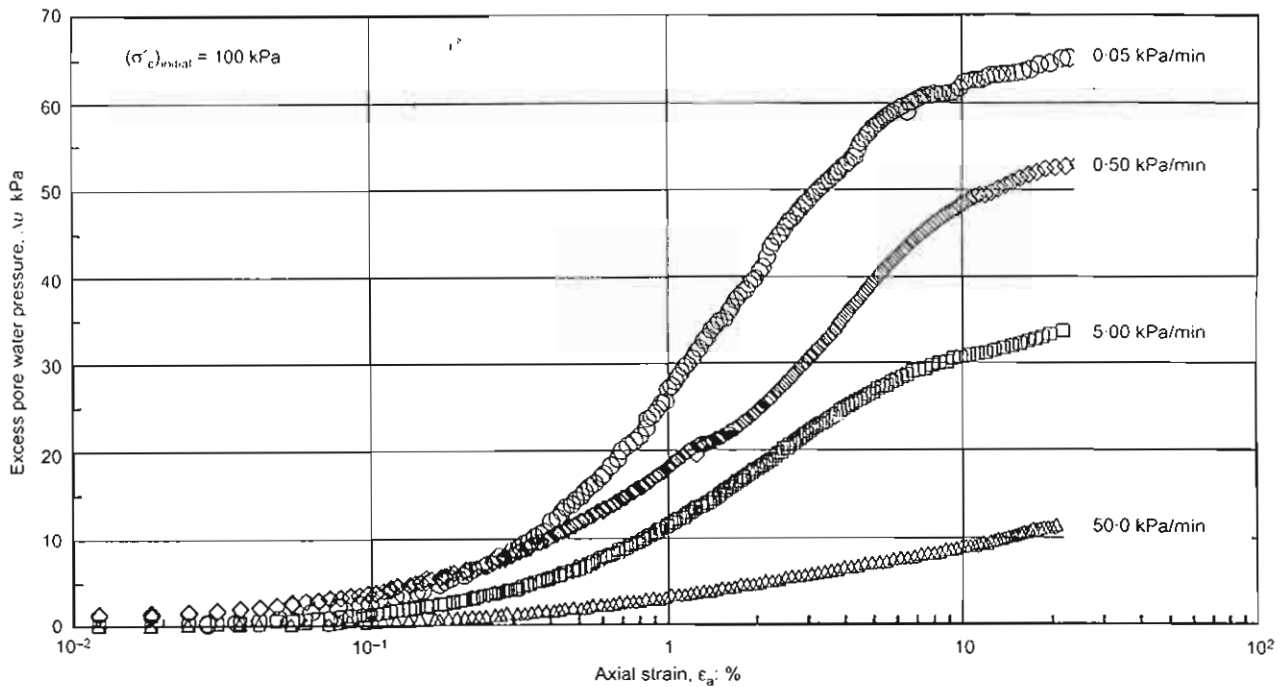


Fig. 2. Development of excess pore water pressure; $(\sigma'_c)_{\text{initial}} = 100 \text{ kPa}$

probably the effective stress change. Therefore the E - ϵ_a curve at large strains obtained from the triaxial compression test conducted at a specific initial consolidation stress is accurate enough for practical use.

Fig. 3 shows the variation of the secant Young's modulus during load-unload-reload cycles of a cyclic loading test. The peak-to-peak deviator stress of about 80 kPa was applied at a slow load frequency of 0.00025 Hz. This was equivalent to a stress rate of about 0.04 kPa/min. The origins for determination of the secant Young's modulus were shifted to the point of stress reversal, as schematically drawn in Fig. 4. For first monotonic loading (marked 'Load 1' in the figure), the degradation curve of E is very similar to that obtained from the triaxial

compression test performed at a stress rate of 0.05 kPa/min (see Fig. 1(a)). When the first stress reversal occurred, the subsequent secant Young's modulus (marked 'Unload 1') at $\epsilon_a \approx 0.02$ – 0.2% increased considerably. For the first few loading cycles the E - ϵ_a curves in this strain range were not much altered. However, for the last loading cycle, when the sample approached failure, E obviously decreased. This is because of the build-up of excess pore water pressure. The effective mean stresses $[p' = (\sigma'_1 + 2\sigma'_3)/3]$ at the beginning of each stress reversal are shown in the figure. When strains become greater than 1% the degradation curves almost merge, and the effect of effective stress change seems to diminish.

Note that the effect of stress reversal on the subsequent

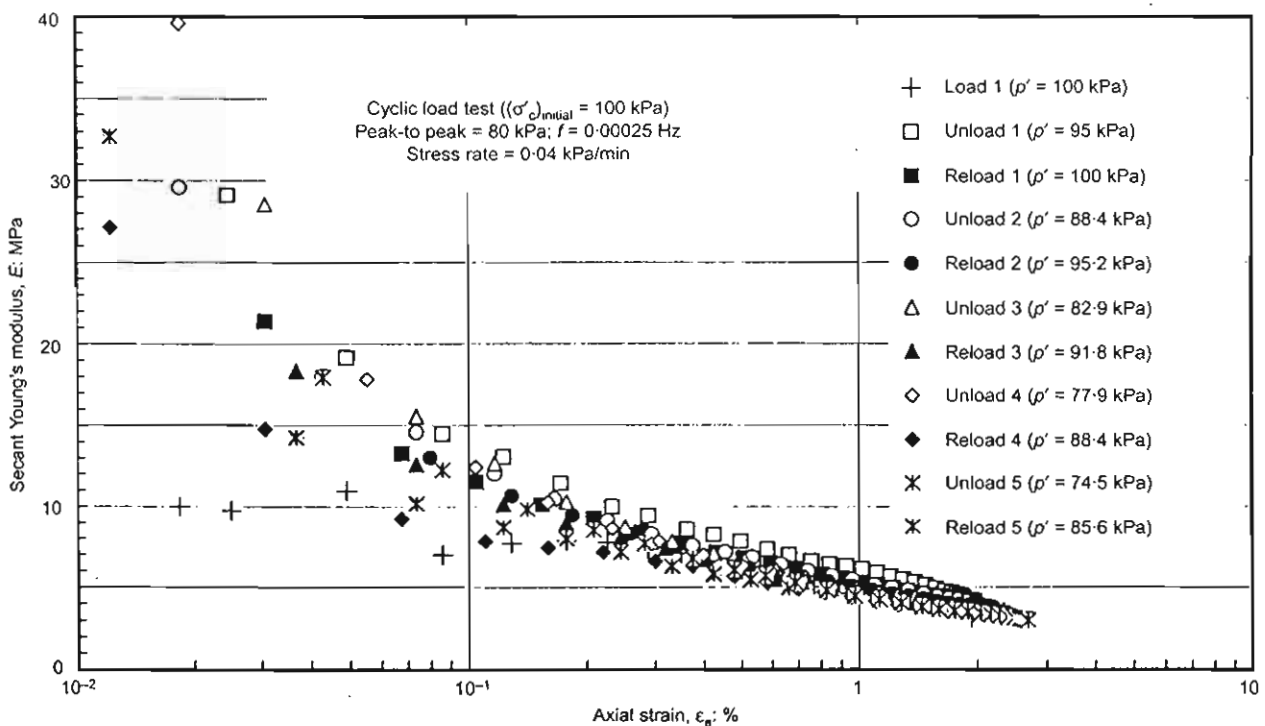


Fig. 3. Secant Young's modulus during load-unload-reload cycles

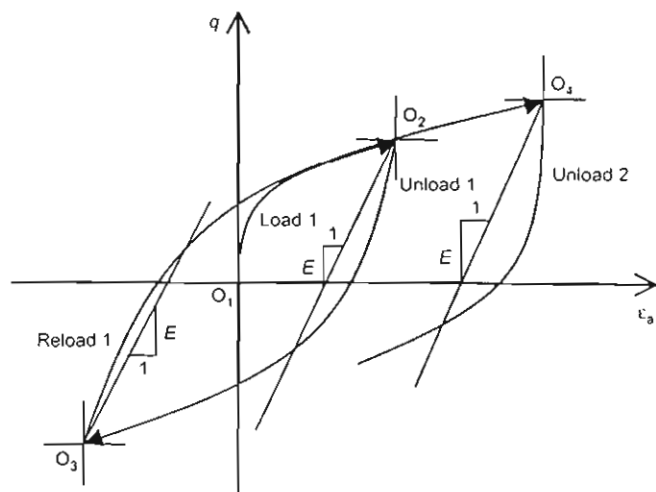


Fig. 4. Secant Young's modulus of the load-unload-reload cycles during a cyclic loading test

secant Young's modulus might suppress the effect of loading rate. This is because the stiffness curves following the first stress reversal are similar to that obtained from the staged cyclic load test [solid line in Fig. 1(a)]. Therefore the effect of stress rate concluded from the results of cyclic loading tests will not be clearly seen, as reported by Shibuya *et al.* (1995).

In general, at small to medium strain levels, the effect of effective stress change is pronounced and cannot be neglected. However, when the strains become larger, the strain level dependence characteristic of E overcomes the effect of effective stress change, and the initial consolidation stress dominates the stiffness degradation obtained at large strains.

CONCLUSIONS

The secant Young's modulus at moderate strains ($\epsilon_s \approx 0.02-0.2\%$) was influenced by the stress rate: the faster the loading rate, the larger the secant Young modulus. When the effective stress at the origin (used for determination of the secant Young's modulus) decreased, the secant Young's modulus at moderate strains also decreased. However, at large strain levels, the strain dependence characteristic of the secant Young's modulus overcame the effects of loading rate and effective stress change. The degradation of E at large strains was dependent almost entirely on the initial consolidation stress.

ACKNOWLEDGEMENT

The work presented in this paper forms part of a project on 'Research and development on natural disaster reduction for sustainable development' supported by a grant from the Thailand Research Fund (TRF).

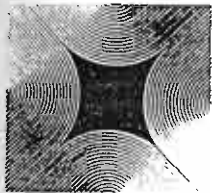
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ภาคผนวก ข

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MS # 01042

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File No.: 01042
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Shear modulus and damping of Soft Bangkok Clays

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Shear modulus and damping of Soft Bangkok Clays

By Supot Teachavorasinskun, Pipat Thongchim and Panitan Lukkunaprasit

Abstract: The shear modulus and damping ratios of the undisturbed Bangkok clay samples were measured using the cyclic triaxial apparatus. Although abundant of literature on this topic can be referred, selection of the most suitable empirical correlation for a seismic analysis cannot be done unless some actual data is obtained. The apparatus used can measure the stress-strain relationships from strain levels of about 0.01%. The equivalent shear modulus measured at these shear strain levels was account for about 80% of the value computed from the down-hole shear wave velocity test. The degradation curves of the equivalent shear modulus fell in ranges, which were in close accordance to that reported in the literature, for clay having similar plasticity. Damping ratios varied from about 4-5% at small strains (0.01%) to about 25-30% at large strains (10%). The effects of load frequency and cyclic stress history were lightly investigated. Increase in load frequency from 0.1 to 1.0 Hz, though play no influence on the shear modulus characteristic, caused slight decrease in damping ratios. The effects of the small amplitude cyclic stress history on the subsequently measured shear modulus and damping ratio were almost negligible if the changes in void ratio were properly taken into account.

Key Words: Soft clay, Shear modulus, Damping ratio, Cyclic triaxial test, Cyclic stress history

Introduction

Bangkok, though located at remote distances from active seismic sources, has been hit by more than 20 earthquakes in the past (Nutalaya *et al.* 1985). Large earthquakes at far to moderate epicentral distances (Warnitchai *et al.* 2000) caused most of the events observed. To access the level of the ground motion in Bangkok, the needs for the cyclic properties of the underlain soft deposit are therefore arisen. The two most fundamental soil properties considered are the equivalent shear modulus, G_{eq} , and damping ratio, D . Although these two properties have been thoroughly investigated by a number of researchers, for examples, Hardin and Drnevich (1972), Kokusho *et al.* (1982), Jardine *et al.* (1984), Shen *et al.* (1985), Alarcon-Guzman *et al.* (1989), Tatsuoka and Shibuya (1992) and Tanizawa *et al.* (1994). It is still necessary that the actual laboratory test data using samples of Bangkok clay being obtained to guide the selection of the empirical correlation. The paper presents the results of the cyclic triaxial tests performed on Soft Bangkok clay with emphasizing on their shear modulus and damping properties. It aims to provide the geotechnical engineer with additional database on the shear modulus and damping ratio of soft clay.

Equipment, Material and Testing Procedure

A computer-based-controlled cyclic triaxial apparatus was employed. The servo-hydraulic loading system enables the axial load to be applied in a cyclically manner up to a frequency of about 20 Hz. The loading system is directly installed on the top platen of the triaxial cell as shown in Fig. 1, therefore, there is no need for the load reaction frame. The axial load cell was placed inside the triaxial cell and being used as a feedback in the loading control system. Two displacement transducers, resolution of 0.01 mm and 1.0 mm, were used to measure the axial deformation. The position of the smaller displacement transducer can be adjusted from outside, so that it can be removed when test continues beyond its capacity.

Undisturbed samples were collected from three sites in Bangkok. The general subsoil profiles at those three sites are schematically depicted in Fig. 2. The sites *CU* (*Chulalongkorn University Site*) and *MU* (*Mahidol University Site*) are located in the center of Bangkok, while the *KU* (*Kasetsart University Site*) site is about 15 km north of Bangkok. Table 1 summarizes the general physical properties of the soil used.

The general testing procedure is outlined in Fig. 3. The sample was first trimmed to have a diameter of 35 mm and height of 70 mm. After saturation was ensured, it was undergone consolidation to the desired isotropic consolidation pressure. Note that the *B*-value was checked immediately after completion of application of differential suction. The sample was generally left under high back pressure (200 kPa) and confining stress for another one day. Consequently, the degree of saturation attained a higher level at the beginning of cyclic shearing. The staged cyclic shearing was done at the load frequency of 0.1 Hz and 1.0 Hz.

The Equivalent Shear Modulus

A set of degradation curves of the equivalent shear modulus, G_{eq} , against the single amplitude shear strain, $\Delta\gamma_{SA}$, is shown in Fig. 4. By assuming the value of poisson's ratio (ν) to be 0.5, the shear strain was derived from the axial one. The G_{eq} defined as the ratio between the single amplitudes of the shear stress and shear strain (see Fig. 4) was then obtained.

With the present equipment arrangement, the smallest $\Delta\gamma_{SA}$ reliably measured is in the order of about 0.01%. As a consequence, the shear moduli measured at these strains are only account for about 80% of the values found in the literature (Hardin and Black 1968 and Ashford *et al.* 1997). The empirical equation proposed by Hardin and Black (1968) was adopted for computing the maximum shear modulus, G_{max} , because it gives a comparable

value to that obtained from the down-hole seismic test performed at *CU* site by Ashford *et al.* (1997). It should be noted from the figure that the effect of load frequency on G_{eq} is very small in all cases

The effect of the initial confining stress on the G_{eq} at $\Delta\gamma_{SA} \cong 3 \times 10^{-2}\%$ is shown in Fig. 5. Based on the results of tests performed on the *MU* samples and by adopting the power law of confining stress, namely $G_{eq} \propto \sigma_c^m$, the parameter m of about 0.6 is obtained. The values of the corresponding G_{eq} of the other samples are also plotted. There is almost indiscernible site effect on the G_{eq} measured at this strain level.

The normalized shear modulus degradation curves, $G_{eq}/G_{max} \sim \Delta\gamma_{SA}$, of all tests fall into quite a narrow band (Fig. 6). Note that G_{max} is obtained from the equation proposed by Hardin and Black (1968). The curves from (Vucetic and Dobry 1991) for clayey soils having plasticity index of 15 – 50% were reproduced and plotted in the figure for the purpose of reference. If a hyperbolic equation is used to fit the test results, the equation shown below can give the best fitted;

$$\frac{G_{eq}}{G_{max}} = \frac{1}{1 + \Delta\gamma_{SA}/0.0012}$$

The coefficient of 0.0012 represents the parameter called reference strain, γ_r , defined by Hardin and Drnevich (1972).

Damping property of Bangkok clays

Damping ratios, computed according to the definition given in Fig. 4, are plotted against the single amplitude shear strain in Fig. 7. The samples tested under loading frequency of 1.0 Hz ($f = 1.0$ Hz) yield smaller damping ratios than those tested with $f = 0.1$ Hz. Although this

contradicts to what generally reported by Vucetic and Dobry (1991), the effect of rate of loading on the damping of clay has not been conclusive. Shibuya *et al.* (1996) adopted the Masing's rule to formulate the hysteresis loops of clay at various rates of loading based on the monotonic test results. Decrease in damping ratios with increase in rates of loading has been predicted. The results shown in Fig. 7 support the above mentioned model. Nevertheless, such slight decrease in damping ratios due to increase in load frequency may not play any influence on their practical application, as can be from the figure that all data are located inside the boundary lines reported by Vucetic and Dobry (1991).

The effect of the initial confining pressure was summarized in Fig. 8. The damping ratios obtained at $\Delta\gamma_{s,t}$ of 0.1 % and 1% are plotted against the load frequency and the initial confining pressure. At strain of 1%, samples tested with higher initial confining stress obviously exhibit higher values of damping.

Effects of the small amplitude cyclic stress history

Bangkok subsoil is believed to experience some extents of the cyclic stress history due to the hits of small earthquakes in the past. The effect of small amplitude cyclic stress history, if existed, must therefore be clarified. In the present study, the 50 and 100 cycles of cyclic loads with $\Delta\gamma_{s,t}$ of 0.2-0.3% were used to simulate the cyclic stress history on the sample (see Fig. 3). The G_{eq} obtained before and after application of cyclic stress history is shown in Fig. 9 and 10 for samples tested under load frequency of 0.1 and 1.0 Hz, respectively. Note that the changes in void ratio were taken into account; namely by normalizing the G_{eq} by a function

$$F(e) = \frac{(2.97 - e)^2}{1 + e}. \text{ The degradation curves of the normalized equivalent shear modulus are}$$

almost insensitive to the applied cyclic stress history. Similarly, its effect on the damping ratio is very small as depicted in Fig. 11.

Conclusions

A set of information on the shear modulus and damping property of a soft to medium clay (Bangkok clay) were provided. The effects of the loading frequency, initial consolidation pressure and small cyclic stress history were investigated. The equivalent shear modulus was found insensitive to the change in load frequency ($f = 0.1$ and 1.0 Hz) and the small cyclic stress history. While damping ratio slightly decreased as the load frequency increased.

Acknowledgments

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Table 1 General physical properties of the tested clays.

Sites	Depth of sampling (m)	w_n (%) ¹⁾	LL (%) ²⁾	PI (%) ³⁾	γ_t (kN/m ³) ⁴⁾
<i>MU</i>	4.5-6.0	53-60	59-67	29-30	16
<i>CU</i>	6.0-7.0	60-70	80-85	40-45	16
<i>KU</i>	9.0	60-65	80-85	40-45	16

1) Natural water content, 2) Liquid limit, 3) Plasticity index, 4) Total unit weight

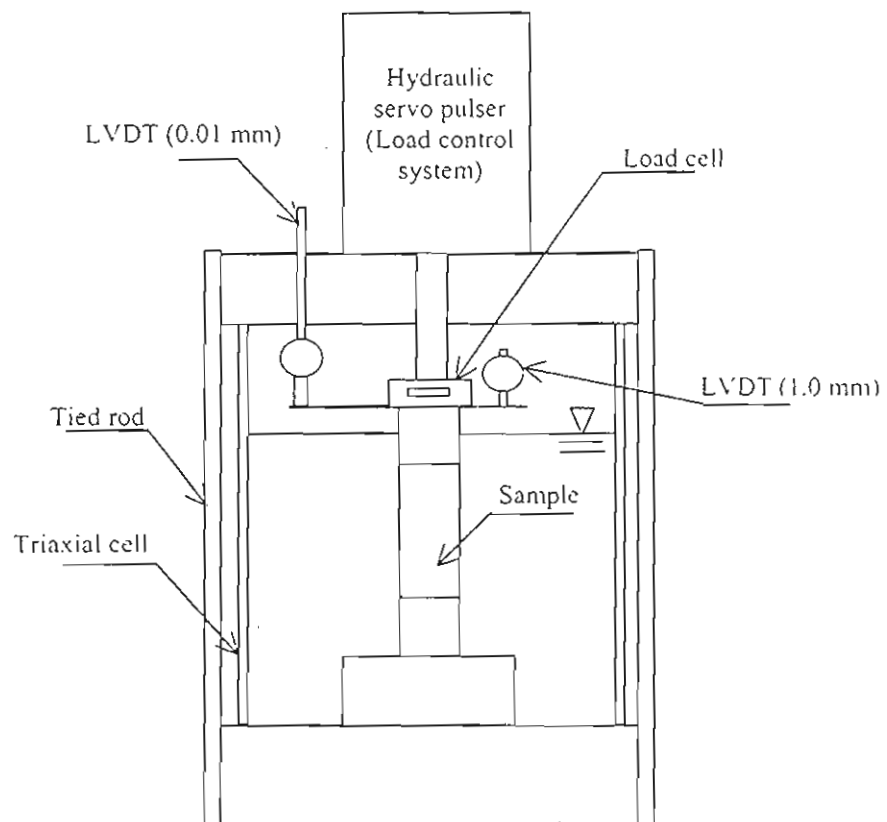


Fig. 1. Location of load and displacement transducers used in the triaxial equipment

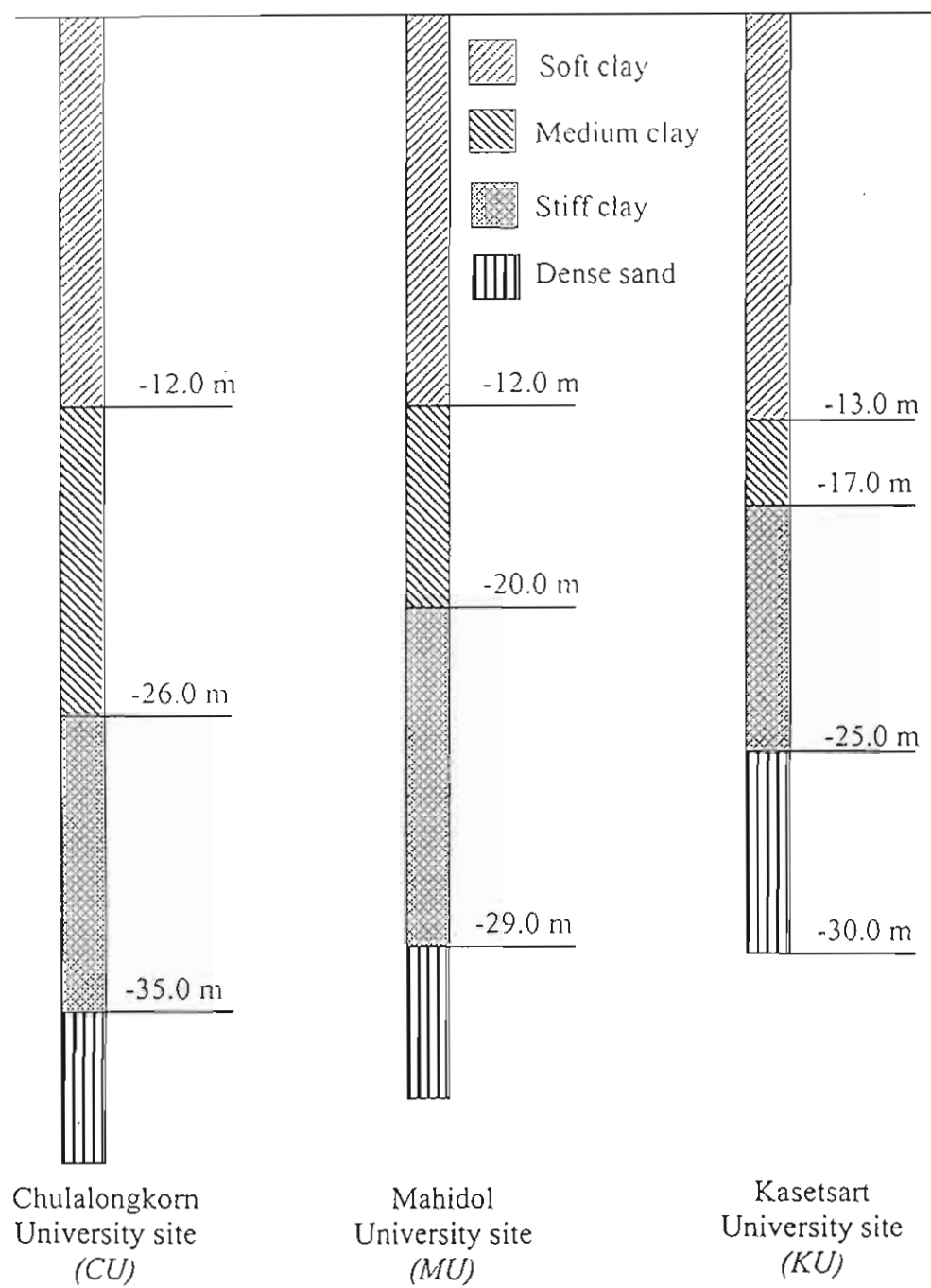


Fig. 2. Simplified soil profiles at three sampling sites in Bangkok

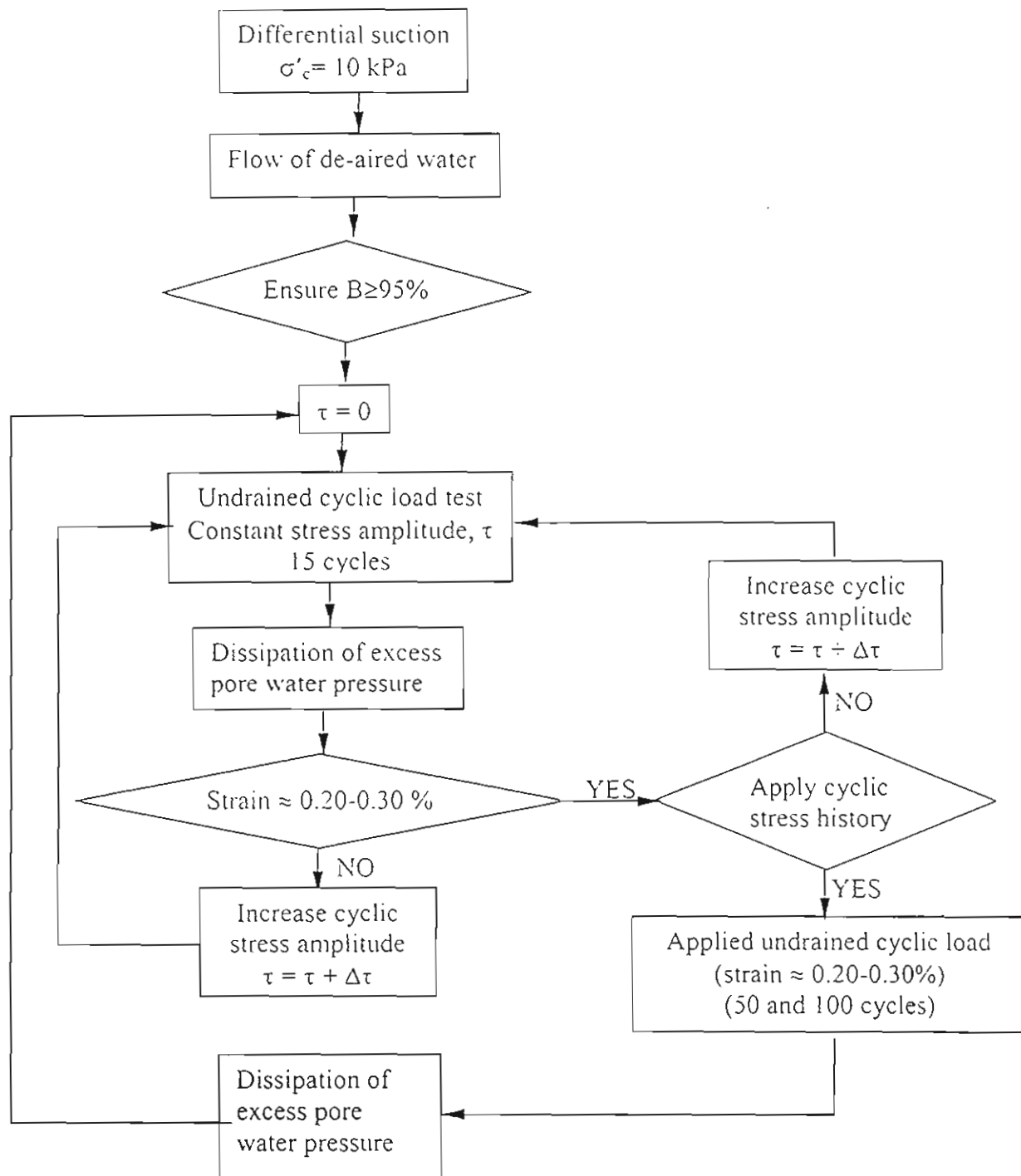


Fig. 3. Flow chart outlining the testing procedure

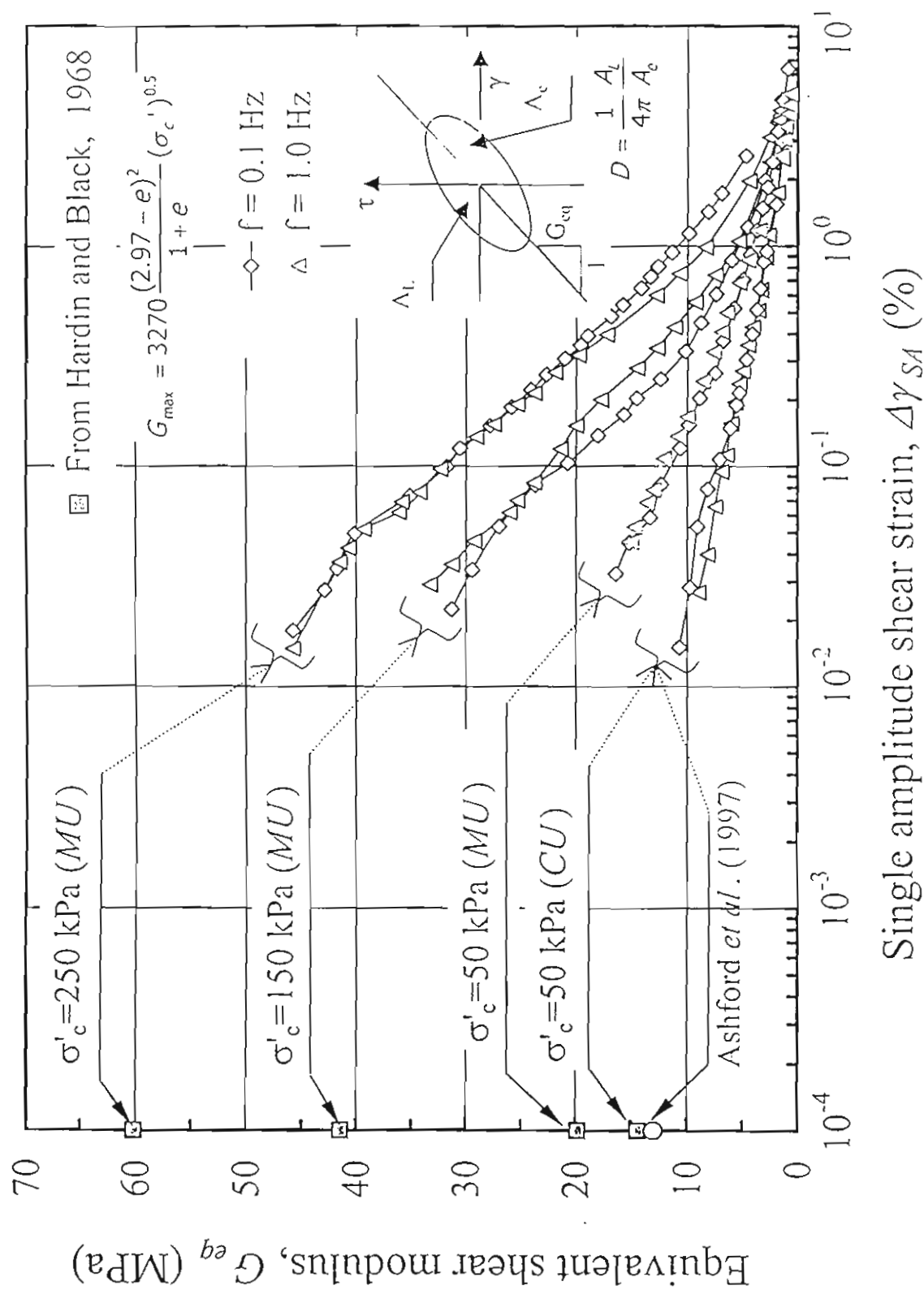


Fig. 4 Degradation curves of the equivalent shear modulus

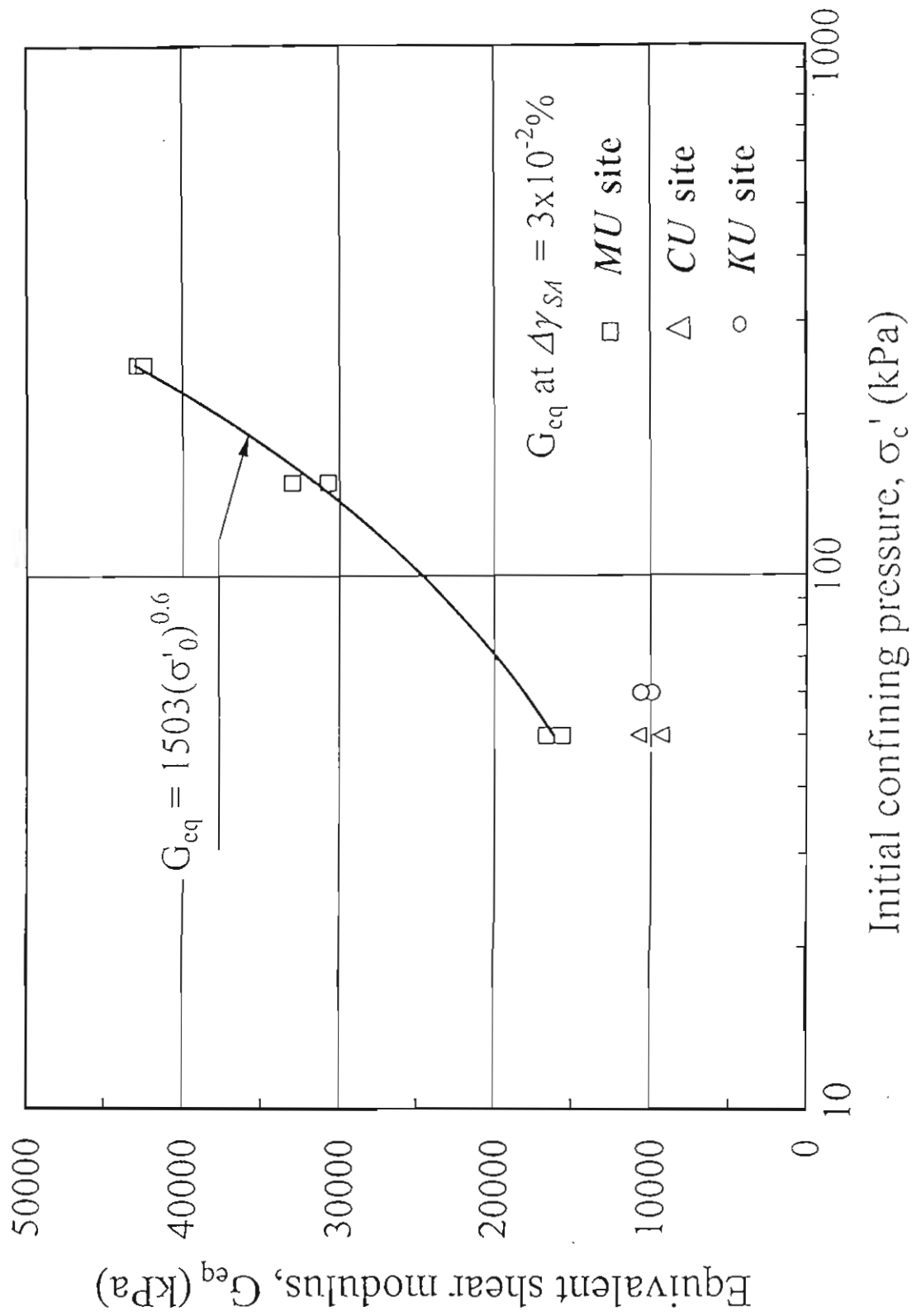


Fig. 5 Effect of the initial confining stress on G_{eq}

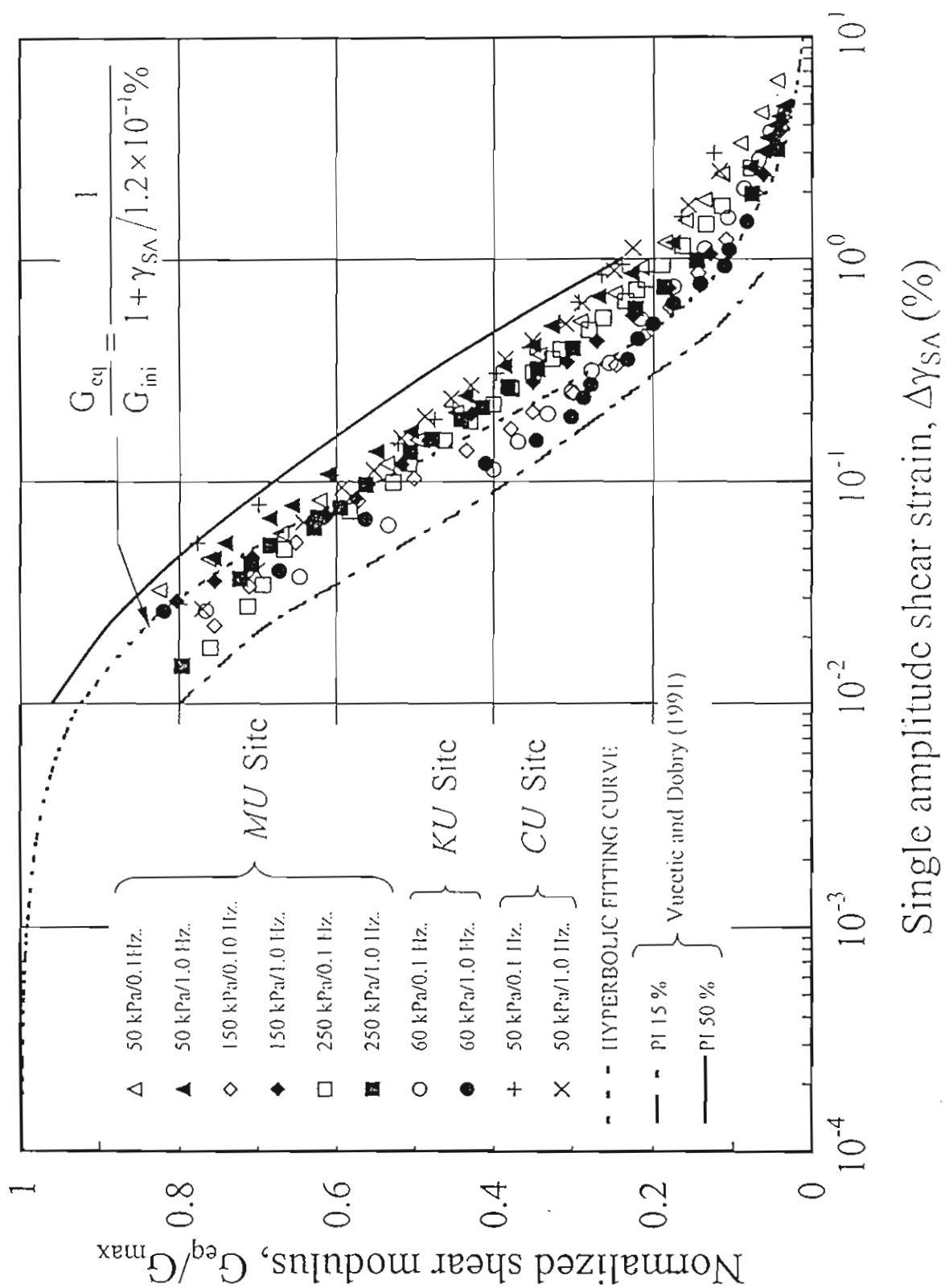


Fig. 6 Normalized shear modulus degradation curves

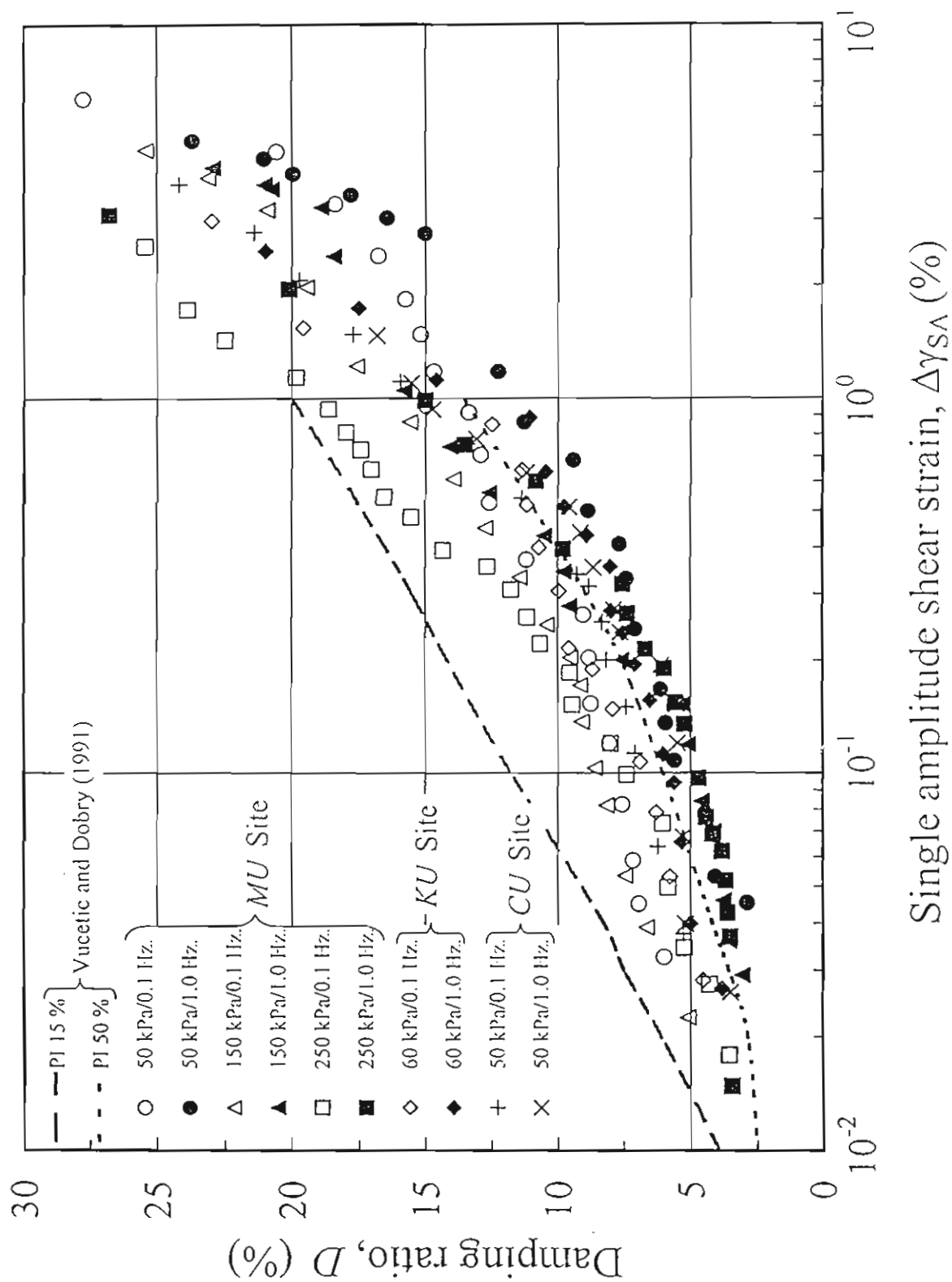


Fig. 7 Relationships between the damping ratios and shear strain

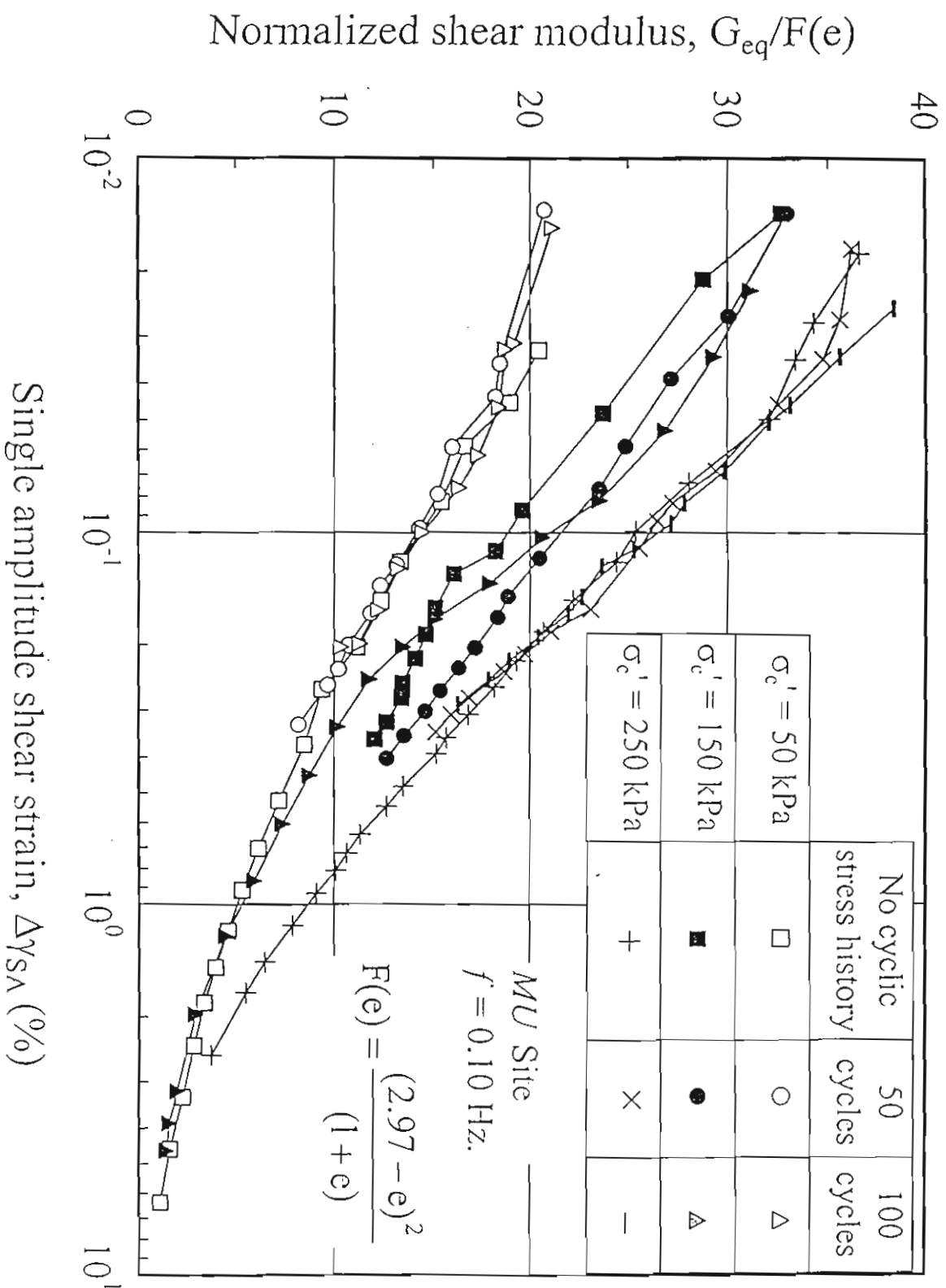


Fig. 9 $G_{eq}/F(e)$ obtained from samples with and without cyclic stress history ($f = 0.1$ Hz)

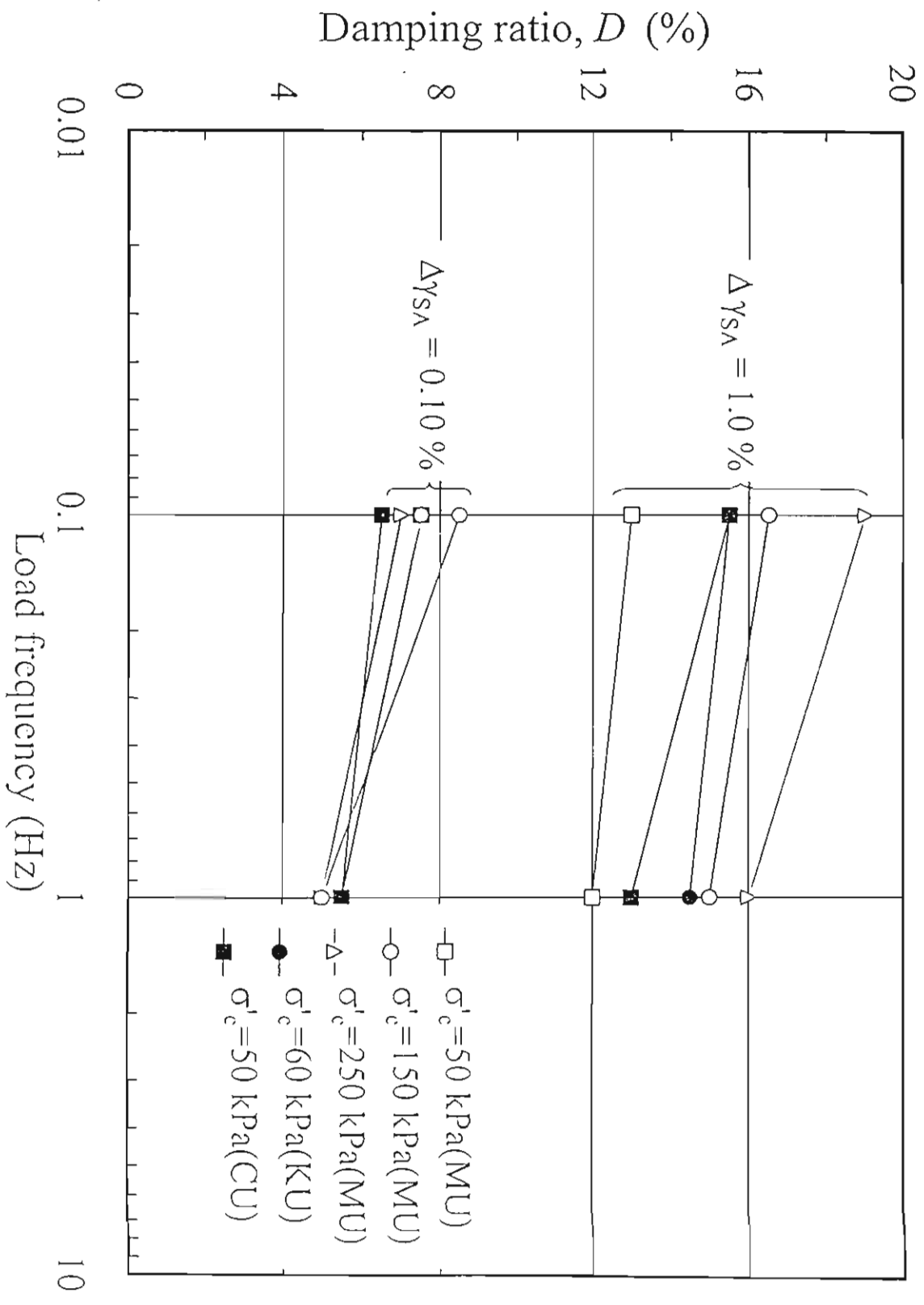


Fig. 8 Effects of load frequency and initial confining stress on d_v

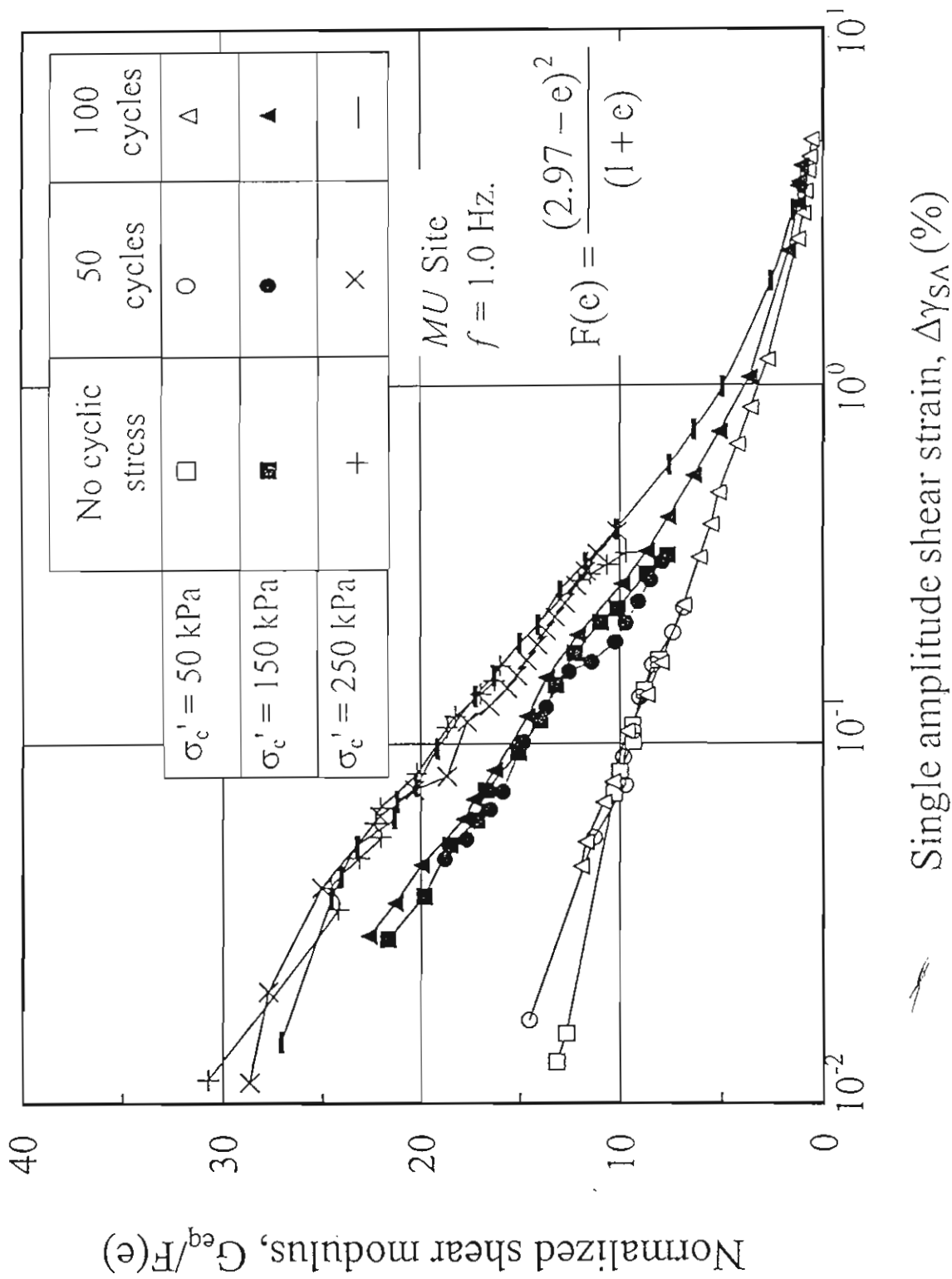
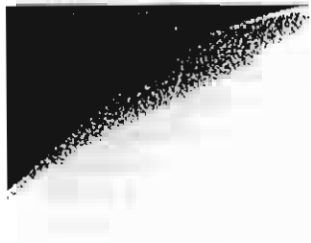


Fig. 10 $G_{eq}/F(e)$ obtained from samples with and without cyclic stress history ($f = 1.0$ Hz)

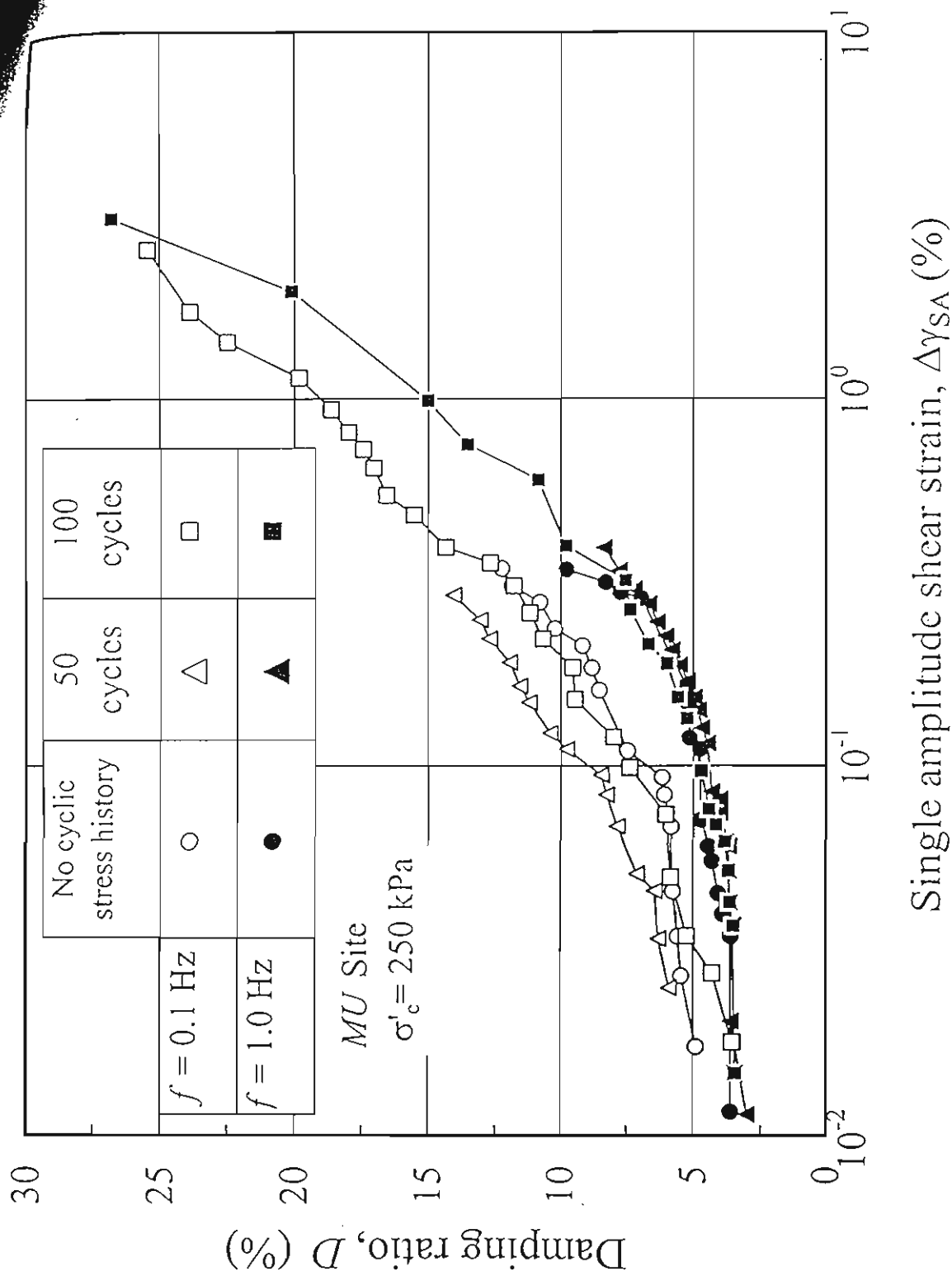


Fig. 11 Effect of cyclic stress history on damping ratios

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14 March 2002

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Dear Dr Tospol,

Paper No. ES/01/406/SK

Authors: T Pinkaew, P Lukkunaprasit & P Chatupote
Title: Seismic effectiveness of tuned mass dampers for damaged reduction of structures

Your paper has been reviewed and for your information, the review report is enclosed. As you can the reviewer is in favour of publication but made a number of comments and suggestions. You are invited to attend to these and revise the paper and I shall be pleased to receive your revised paper (2 copies + disk) for further consideration.

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1. The paper is well written.
2. Before eq. 2 reference to the paper by Fajfar shall be given.
3. p.10: the term 'deteriorated' and 'deterioration' of TMD effectiveness used twice on this page shall not be used, the authors shall refer to 'decreased' etc. effectiveness.
4. References: check if the word 'Handcock tower' is correct in Ref. 1, 'Hancock'?
5. Table 1: the mass of the TMD is ca. 30% of the mass of the structure: check the masses, the building mass is ca 180,000 Mg (tons)
6. Fig. 2: in the caption or the fig. It shall be explained what dotted and the full lines are.
7. In the captions of Figs. 3 to 9 it shall be indicated that these are valid for a harmonic ground excitation with a frequency ofHz and that the eigenfrequency of the structure is ...Hz, instead of that, the frequency ratio may be given
8. Caption of Fig. 3: it shall be mentioned that this is an elastic response with zero damage
9. Caption of Fig. 4: it shall be mentioned that this is an inelastic response with damage index of ... at the end of the harmonic excitation, it would also be interesting to know after how many cycles the damage index of 1 has been reached
10. Define abbreviations in captions or in separate section: displacement ratio; displacement reduction, normalized PGA; normalized energy dissipations
11. Figs. 8 and 9 and 12 and 13: It would be useful if the points with $DI=1$ could be marked as 'failure with TMD' and 'failure without TMD'
12. In the captions of Figs. 10 to 13 it shall be indicated that these are valid for a far-field earthquake excitation from Mexico City.
13. The time history and corresponding response spectrum of the far-field earthquake from Mexico City shall be shown. In the text body the normalization of the accelerogram should also be mentioned briefly, if missing. The eigenfrequency of the building shall be indicated in the response spectrum.
14. In the discussion it should be mentioned that the effectiveness of the TMD is decreasing with increasing damping (comment: in the analysis a damping ratio of 2% has been assumed for earthquake excitation, which is a rather low value), with increasing detuning (comment: in an R.C. building stiffness will also decrease with increasing damage index and therefore during the inelastic response the TMD will be detuned) and with increasing damage index (due to increased hysteretic damping).
15. The definition of the storey restoring force $Q(t)$ should be given.
16. It would also be useful if a statement could be made about the effectiveness of a TMD in the case of a broad-band seismic excitation.

12.3.2002

Paper title:

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Number of pages: 16 (text) + 1 (table) + 13(figures)

Number of tables: 1

Number of figures: 13

SEISMIC EFFECTIVENESS OF TUNED MASS DAMPERS FOR DAMAGE REDUCTION OF STRUCTURES

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Abstract

The effectiveness of tuned mass dampers (TMD) for control of structures under seismic ground motion is investigated. Since describing the effectiveness of TMD using displacement reduction of the structure is found to be insufficient after yielding of the structure, damage reduction of the structure is proposed instead. Numerical simulations of a 20-storey reinforced concrete building modeled as an equivalent inelastic single-degree-of-freedom (SDOF) system subjected to both harmonic and the 1985 Mexico City (SCT) ground motions are considered. It is demonstrated that although TMD cannot reduce the peak displacement of the controlled structure after yielding, it can significantly reduce damage to the structure. In addition, certain degrees of damage protection and collapse prevention can also be gained from the application of TMD. This is of particular significance

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in view of the current trend toward performance based design in which TMD may be installed to achieve the target performance of buildings designed for seismic loads.

Keywords: tuned mass dampers; seismic excitations; inelastic structures; damage reduction

1. Introduction

The passive tuned mass damper (TMD) is found to be a simple, effective, inexpensive, and reliable means to suppress undesirable vibrations of structures caused by harmonic or wind excitations [1,2]. Under earthquake excitation, which is rather random, its performance, however, greatly depends on the characteristics of ground motion [3-5]. It was found that TMD consequently becomes effective in reducing the seismic response of structures only when the ground motion exhibits narrow band frequency and long duration.

Villaverde and Koyoama [6] studied the influence of TMD on the response of a ten-storey building subjected to the S60E component of the SCT accelerogram, Mexico City earthquake, 1985. Due to the narrow bandwidth and long duration of the ground motion record, they observed a reduction in the peak roof displacement of the building of about 40 per cent when a TMD with modal mass ratio of 0.042 was installed on its top.

Taking into account the possibility of damage to the building during a high intensity earthquake, Soto-Brito and Ruiz [7] studied the influence of ground motion intensity on the effectiveness of TMD. The response of a 22-storey nonlinear frame with a TMD was considered under moderate and high intensities of SCT accelerograms. Their study showed that the effectiveness of TMD in reducing the peak responses of the structure can be

substantially decreased due to nonlinear behavior of the building, which generally occurs under high intensity ground motions.

However, unlike the linear system, describing the effectiveness of TMD using peak response reduction of the nonlinear structures alone seems insufficient [8]. Obviously, this measure cannot account for the effects of accumulated damage due to low cycle fatigue. That is of particular importance for the seismic application where the structure may experience a significant number of nonlinear vibration cycles. Therefore, in such circumstances, TMD is expected to effectively reduce not only the peak response of the structure but also the induced-damage of the structure.

In this study, damage reduction is proposed as an indicator to evaluate seismic effectiveness of TMD with inelastic structures. A 20-storey reinforced concrete building is modeled by an equivalent inelastic single-degree-of-freedom system. The numerical simulations of the building with and without TMD attached on top are performed. Two records of ground motion characterized by a harmonic and a distant earthquake are examined. The inelastic behavior of the structure is introduced by increasing the peak ground acceleration of ground motions. By considering the variation of the energy dissipation in the structure, the TMD's control characteristic and its inability to control high intensity ground motion as found in the literature are explained. Since the structure is excited in the inelastic range, the damage is quantified employing the well-accepted damage index [9]. By comparing the induced-damage indices for the structure with and without TMD, the effectiveness of TMD can be evaluated and explained.

2. An example reinforced concrete building

In this study, a 20-story reinforced concrete office building is considered. The plan and elevation of the building is shown in Fig. 1. The building utilizes a structural system with moment-resisting frames in the longitudinal direction and a coupled moment-resisting frame-shear wall system in the transverse direction. The building is designed only for gravity and wind loads for Thailand with non-ductile reinforcement details. A uniformly distributed live load of $2,500 \text{ N/m}^2$ and a maximum wind pressure of approximately $1,400 \text{ N/m}^2$ are employed. Compressive strength of concrete of 30 MPa and yield strength of reinforcement of 40 MPa are used. The cross-sections of the shear walls and beams are $0.30 \times 8.00 \text{ m}$ and $0.30 \times 0.60 \text{ m}$, respectively, while the dimensions of the columns vary from $0.95 \times 0.95 \text{ m}$ at the first floor to $0.45 \times 0.45 \text{ m}$ at the roof floor. The program IDARC [10] is used to model the structure. The fundamental period of the structure, determined from analysis in the transverse direction, is found to be 2.13 seconds. Pushover analysis using an inverted triangular load is also performed. Fig. 2 plots the roof displacement of the structure against the base shear of the structure. It should be noted that response due to designed wind load is well within the elastic range of the curve.

3. An equivalent inelastic SDOF

The 20-story building outlined in previous section is modeled by an equivalent inelastic single-degree-of-freedom (SDOF) system. The nonlinear pushover characteristics of the building are ascertained and then its load-deformation characteristic is replaced by the elastic-perfectly plastic behavior of the SDOF system. Although there exist many

formulations in the literatures for selection of the equivalent inelastic SDOF, the formulation proposed by Fajfar and Fischinger [11] is adopted in this study.

The equation of motion that governs the response of the structure when subjected to ground excitation can be expressed as

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{Q}(t) = -\mathbf{M}\mathbf{r}\ddot{x}_g(t) \quad (1)$$

where

$\mathbf{M}$ and $\mathbf{C}$ are, respectively, the mass and damping matrices of the structure;

$\ddot{\mathbf{u}}(t)$ and $\dot{\mathbf{u}}(t)$ are, respectively, the acceleration and velocity vectors of the structure relative to the ground;

$\mathbf{Q}(t)$ is the storey restoring force vector of the structure;

$\mathbf{r}$ is the influence coefficient vector which represents the displacement vector, $\mathbf{u}(t)$, resulting from a unit support displacement and

$\ddot{x}_g(t)$ is the acceleration of the input ground motion.

The base shear force of the structure is determined from $V(t) = \{\mathbf{1}\}^T \mathbf{Q}(t)$.

Using the transformation of the roof displacement of the structure, $u^*(t)$, to the displacement of the SDOF, $x(t)$, by

$$x(t) = \frac{\Phi^T \mathbf{M} \Phi}{\Phi^T \mathbf{M} \mathbf{r}} u^*(t) \quad (2)$$

in which Φ is the assumed shape vector of the structure normalized with respect to $u^*(t)$, the above equation of motion of the structure, Eq. (1), can be rewritten as

$$m\ddot{x}(t) + c\dot{x}(t) + q(t) = -m\ddot{x}_g(t) \quad (3)$$

where

$m = \Phi^T \mathbf{M} \mathbf{r}$ is an equivalent mass;

$c = \Phi^T \mathbf{C} \Phi \cdot \frac{\Phi^T \mathbf{M} \mathbf{r}}{\Phi^T \mathbf{M} \Phi}$ is an equivalent damping;

$q(t) = \Phi^T \mathbf{Q}(t)$ is an equivalent restoring force and

$\ddot{x}(t)$ and $\dot{x}(t)$ are the acceleration and the velocity of the equivalent SDOF.

Based on the load-displacement relationship of the structure obtained from pushover analysis with an inverted-triangle load distribution as shown in Fig. 2, the corresponding properties of the equivalent elastic-perfectly plastic SDOF previously mentioned can be computed as listed in Table 1 and plotted as the dotted-line in the same figure.

4. Earthquake ground motions

Two different ground motions, characterized by harmonic acceleration and distant earthquake acceleration, are employed as the input ground motions. Both are scaled to have various peak ground accelerations (PGA) in order to study the influence of the degree of inelasticity in the structure on the effectiveness of TMD. Harmonic ground motion is generated for 50 seconds from a constant amplitude sine wave with the same period as that of the structure, while the acceleration record at the SCT station of the 1985 Mexico City earthquake is used to represent distant earthquake ground motion. The dominant period of this acceleration record is very close to that of the structure. These intentionally lead to extreme conditions where the structure vibrates in resonance with the excitations and consequently TMD is expected to be effective for suppression of both displacement and energy of the structure.

5. Damage quantification

For earthquake excitation, it is generally necessary to permit some degree of induced-damage in the structure, otherwise the design would be too costly. However, the damage should be kept below a certain amount in order to avoid collapse.

To investigate the effectiveness of TMD in reducing damage to the structure induced by ground excitation, the damage index as suggested by Park et al. [9] is adopted for the system Eq. (1), which can be globally described by

$$DI = \frac{u_m^*}{u_u^*} + \beta \frac{\int dE}{u_u^* V_y} \quad (4)$$

where

u_m^* and u_u^* are the maximum and the ultimate displacements of the structure at roof, respectively;

$\int dE$ is the cumulative hysteresis energy of the structure;

V_y is the base shear at yielding of the structure and

β is the strength deterioration parameter (≈ 0.27 by Ciampoli, et al. [12]).

This damage equation accounts for damage due to maximum inelastic displacement, as well as damage due to cumulative low-cycle fatigue. The value of DI can vary from 0 to 1.0, which corresponds respectively to the damage level of the structure from no damage to collapse [13].

6. Structure with tuned mass damper (TMD)

A TMD is installed on top of the structure to reduce the displacement and the damage induced by ground excitations. For computational convenience, the equivalent inelastic SDOF obtained in Section 3 is employed to represent the 20-story R/C building as outlined in Section 2. Thus the equations of motion describing the structure-TMD interaction can be simply expressed by

$$m\ddot{x}(t) + c\dot{x}(t) + q(t) = -m\ddot{x}_g(t) + c_T\dot{z}(t) + k_T z(t) \quad (5)$$

and

$$m_T\ddot{z}(t) + c_T\dot{z}(t) + k_T z(t) = -m_T [\ddot{x}_g(t) + \ddot{x}(t)] \quad (6)$$

where

m_T , c_T and k_T are the mass, damping, and stiffness of the TMD, respectively;

and $\ddot{z}(t)$, $\dot{z}(t)$, and $z(t)$ are the relative acceleration, velocity and displacement of the TMD.

In the following numerical investigation, a mass ratio of 1.40%, corresponding to an effective mass ratio of 0.03 between the TMD mass and the structure's first-mode generalized mass, is considered. Assuming linear behavior of the controlled structure, the stiffness and damping coefficients of TMD are optimized for harmonic base excitation [14]. This yields the optimal values of the TMD parameters as listed in Table 1 along with the properties of the example structure.

7. Effectiveness of TMD under a harmonic ground motion

The effectiveness of TMD is evaluated under harmonic ground motion. The inability to control high intensity ground motion is demonstrated and explained. The use of damage

reduction of the structure as the effectiveness measure of TMD is introduced instead of using displacement reduction alone.

7.1 Displacement Reduction

Figs 3 and 4 show the typical displacement histories of the structure with and without TMD under harmonic ground motion with PGAs of 0.01g and 0.05g, respectively. The motion frequency is intentionally set to be the natural frequency of the structure. In these figures, the former represents the case where the structure is vibrated within the elastic range and significant displacement reduction of the structure can be gained from the application of TMD, whereas the latter represents the case where the structure is vibrated well within the inelastic range and the displacement reduction of the structure by application of TMD is substantially decreased. To show the influence of degree of inelasticity in the structure on the performance of TMD, Fig. 5 compares the displacement ratio of the structure with and without TMD for various PGAs of ground motion. In the figure, the displacement ratio of the structure is calculated from the ratio of the steady-state displacement of the inelastic structure with or without TMD to that of the corresponding elastic structure without TMD, while the input PGA is represented by the normalized PGA, $|\ddot{x}_g|/(q_y/m)$. To ascertain the effectiveness of TMD in terms of displacement reduction of the structure, Fig. 6 plots the steady-state displacement reduction of the structure due to the application of TMD as a function of normalized PGA. It is obvious from Figs. 5 and 6 that TMD is very effective in reducing the displacement of the structure by as much as 77% when normalized PGA is less than 0.042. This is the case where the structure is vibrated within its elastic range. However, beyond this range, the TMD's effectiveness gradually decreases as the PGA increases and

becomes as small as 10% when normalized PGA is greater than 0.176. These findings coincide with those found by previous studies [4,7] in which the effectiveness of TMD is reported to be substantially deteriorated when the structure's motion is characterized by significant inelastic vibration. This control deficiency can be demonstrated through inspection of the energy dissipation of TMD as shown in Fig. 7. In the figure, the energy dissipation in the structure due to inherent damping, yielding and TMD are normalized by the corresponding kinetic energy of the structure and are plotted against the normalized PGA. The figure shows that TMD slightly loses its effectiveness in dissipating the energy from the structure once yielding occurs in the structure. This is because yielding raises the apparent damping of the structure and consequently disturbs the tuning condition of TMD. Although the dissipation reduction of TMD seems significant in this considered range of PGA, the large amount of dissipation due to yielding causes the contribution of TMD to become a comparatively small portion. As a result, a displacement reduction of the structure of only 10% is obtained from the application of TMD.

7.2 Damage Reduction

It is clear from Figs 5 and 6 that TMD can completely protect the building from yielding up to a normalized PGA of 0.156. Although the displacement reduction in this range indicates deterioration of TMD's effectiveness, this is of particular importance for seismic applications in which the objective of TMD installation is to suppress the damage to the structure rather than its displacement.

Therefore, the following introduces the damage reduction of the structure to describe the effectiveness of TMD instead of the displacement reduction. Employing the

damage model previously outlined, Fig. 8 compares the damage index of the structure with and without TMD under harmonic ground motion, while Fig. 9 shows the damage reduction gained from TMD. In the figure, the damage reduction is calculated from the difference between damage to the structure with TMD and without TMD divided by that without TMD, $(DI_{w/o}-DI_{TMD})/DI_{w/o}$. It is pointed out that, for normalized PGA less than 0.042, the damage reduction of the structure due to TMD is not defined since there is no damage to the corresponding structure without TMD. Unlike the displacement reduction as in Fig. 6, the figure shows complete damage reduction due to TMD for normalized PGA between 0.042 and 0.156. This implies the usefulness of TMD even when the structure is subjected to moderate input ground motion. However, further increasing the PGA causes yielding in the structure with TMD and damage reduction of about 40-70% is obtained for normalized PGA between 0.156 and 0.306. In this range of input PGA, the damage reduction decreases as the PGA increases. It is also observed that, for normalized PGA between 0.306 and 0.443, the computed damage of the structure without TMD exceeds the collapse limit, i.e. $DI>1.0$, while that of the structure with TMD is still within the limit. This corresponds to the case where the TMD can prevent the structure from collapse. Beyond this range of input PGA (normalized PGA greater than 0.443), the structure with TMD also collapses and therefore the application of TMD to the structure provides no benefit.

8. Effectiveness of TMD under a distant earthquake

In this section, ground motion recorded from the distant earthquake in Mexico City (1985) at SCT station in the N-S direction is employed as the input ground excitation. This record

is found to have its predominant period close to the first natural period of the structure. The effectiveness of TMD is again evaluated in terms of displacement and damage reduction of the structure.

8.1 Displacement Reduction

Fig. 10 plots the displacement ratio of the structure with and without TMD against the input PGA of ground motion. In this figure, the displacement ratios are calculated using peak displacement instead of steady-state displacement as in Fig. 5. It can be seen from the figure that TMD is very effective in reducing the peak displacement of the structure when the structure is vibrated within the elastic range. However, its effectiveness gradually decreases as the inelasticity in the structure increases. It is noticed that, for normalized PGA higher than 0.40, the structure with TMD exhibits larger displacement than that without TMD. This leads to negative displacement reduction of TMD as shown in Fig. 11.

8.2 Damage Reduction

Fig. 12 compares the damage indices of the structure with and without TMD under the SCT ground motion for various PGAs, while Fig. 13 shows the damage reduction gained from the application of TMD. The figures reveal that TMD always reduces the damage to the structure for the entire range of PGA considered. Similar to the case of the harmonic ground motion, TMD perfectly protects the structure from damage for normalized PGA less than 0.20, and yields about 20-40% reduction of the damage for normalized PGA from 0.20 to 0.685. It is interesting to note that, for normalized PGA greater than 0.40, TMD provides positive effectiveness in damage reduction of the structure although its effectiveness in terms of displacement reduction is found to be negative (see also Fig. 11). This is because the

damage index of the structure under the SCT ground motion is mainly governed by the energy term, i.e. the second term of Eq. (4), which is effectively reduced by TMD. It should also be noted that, for this ground excitation, TMD prevents collapse for normalized PGA from 0.685 to 0.841. Beyond this range of the input PGA, the structure with TMD collapses and therefore the application of TMD becomes useless.

9. Conclusions

The effectiveness of the TMD for control of the 20-storey reinforced concrete building subjected to both harmonic and the 1985 Mexico City (SCT) ground motions is investigated. Using numerical simulation, the obtained results indicate that the commonly used displacement reduction of the structure fails to prescribe the effectiveness of the TMD when damage occurs in the structure. This is because it does not provide sufficient information on the damage state of the controlled structure which is the main concern in engineering applications.

Therefore, direct use of damage reduction of the structure becomes more relevant and it is employed as the indicator of TMD effectiveness in this paper. Based on the ground motions considered, which have predominant periods close to that of the structure, the TMD is found to provide damage protection for the structure up to a certain level of input PGA. With a higher level of input PGA, it also significantly reduced the damage to the structure, which would suffer substantial damage in its absence. With further increase of input PGA, it is found that the application of TMD can prevent the structure from collapse.

By comparing these damage characteristics to those of the structure with higher yield strength of base shear, this application of TMD is equivalent to an increase in the yield strength of the structure of about 45% and 20% for the harmonic and the SCT ground motions, respectively. This is of practical significance in view of the current trend toward performance based design in which the TMD may be installed to achieve the target performance of buildings designed for seismic loads.

Acknowledgements

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and excitation parameters. *Earthquake Engineering and Structural Dynamics* 1982;
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Table 1 Properties of the structure, the equivalent inelastic SDOF and TMD.

Structure	System, Eq.(1)	$W = 378.0 \text{ MN}$, $V_y = 43.5 \text{ MN}$, $u_y^* = 0.258 \text{ m}$, $u_u^* = 1.68 \text{ m}$, ductility = 6.5
	SDOF, Eq.(3)	$m = 17.9 \times 10^4 \text{ tons}$, $\omega = 2.95 \text{ rad/s}$, $\xi = 0.02$ $q_y = 27.4 \text{ MN}$, $x_y = 0.175 \text{ m}$, $x_u = 1.14 \text{ m}$
Tuned Mass Damper (TMD)		$m_T = 5.37 \times 10^4 \text{ tons}$, $k_T = 4.41 \times 10^5 \text{ kN/m}$, $\omega_T = 2.87 \text{ rad/s}$, $\xi_T = 0.105$

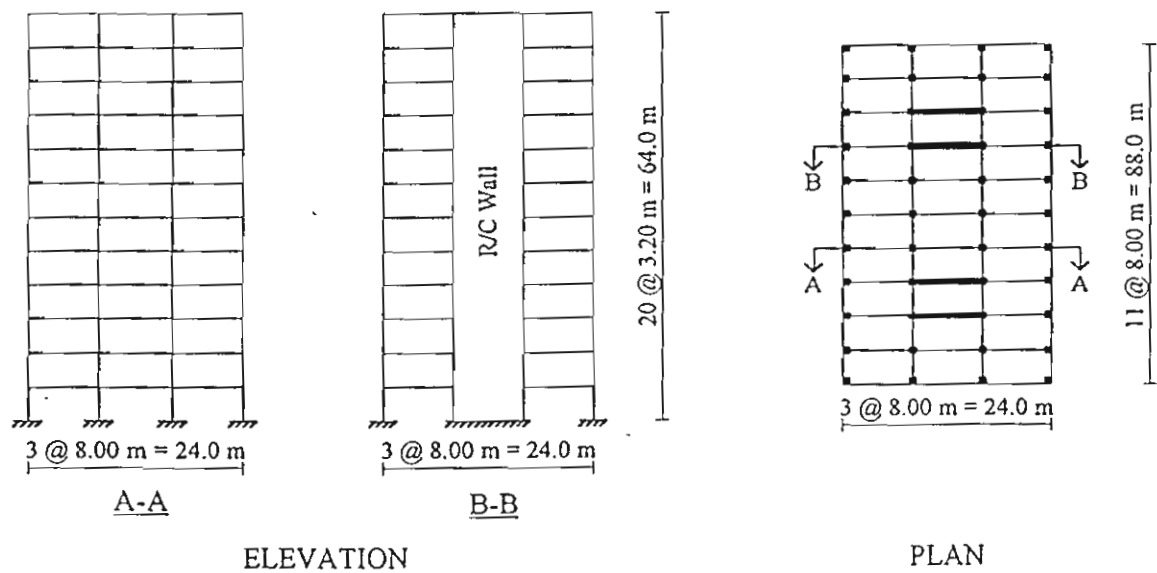


Figure 1 Elevation and plan of a 20-storey reinforced concrete building

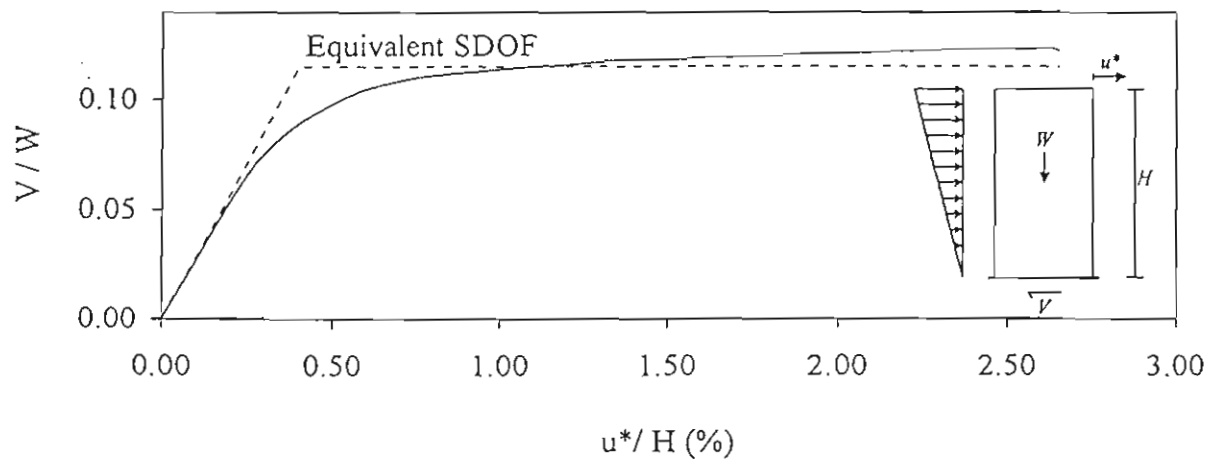


Figure 2 Pushover curve of structure

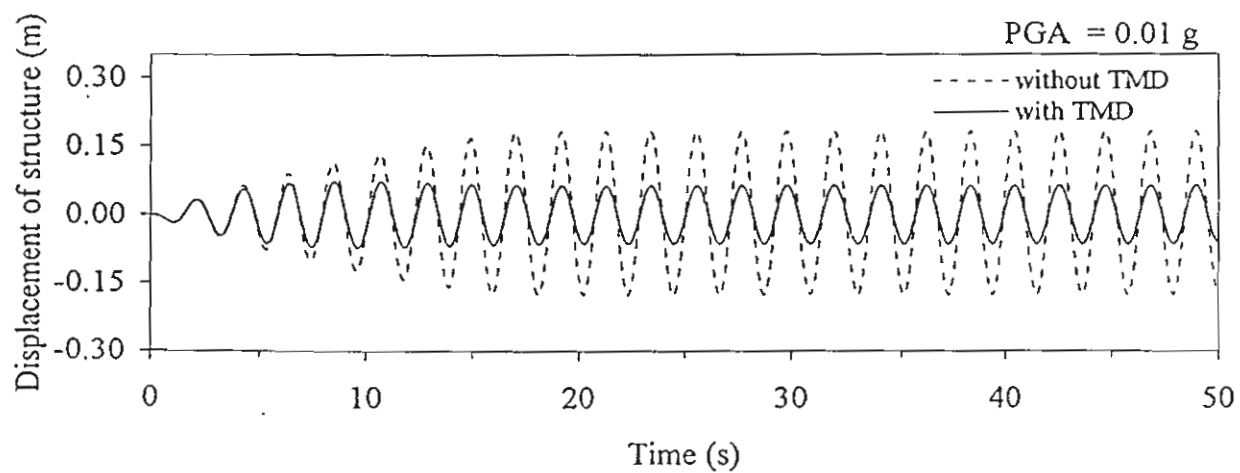


Figure 3 Displacement histories of structure with and without TMD under harmonic ground motion with PGA of 0.01g

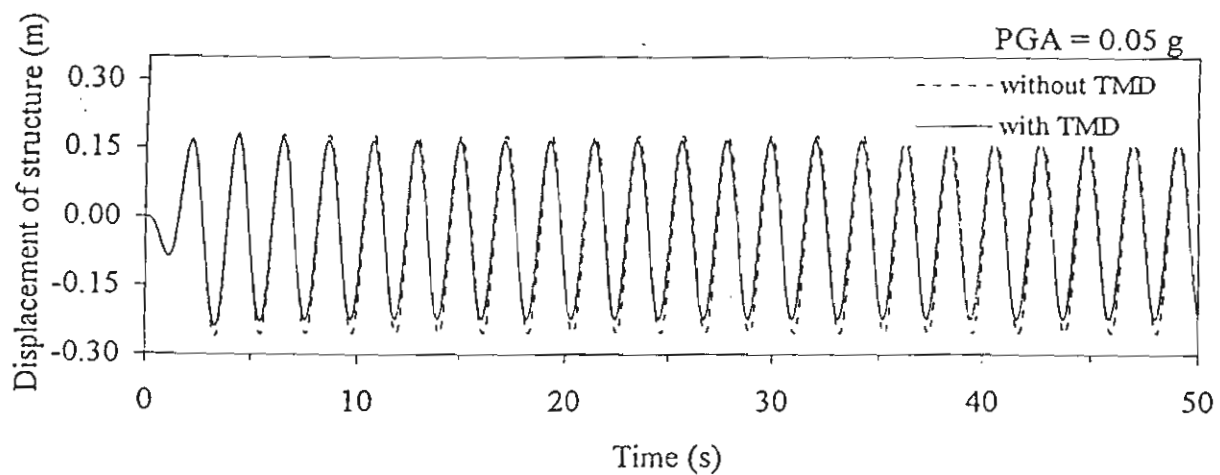


Figure 4 Displacement histories of structure under harmonic ground motion with PGA of 0.05g

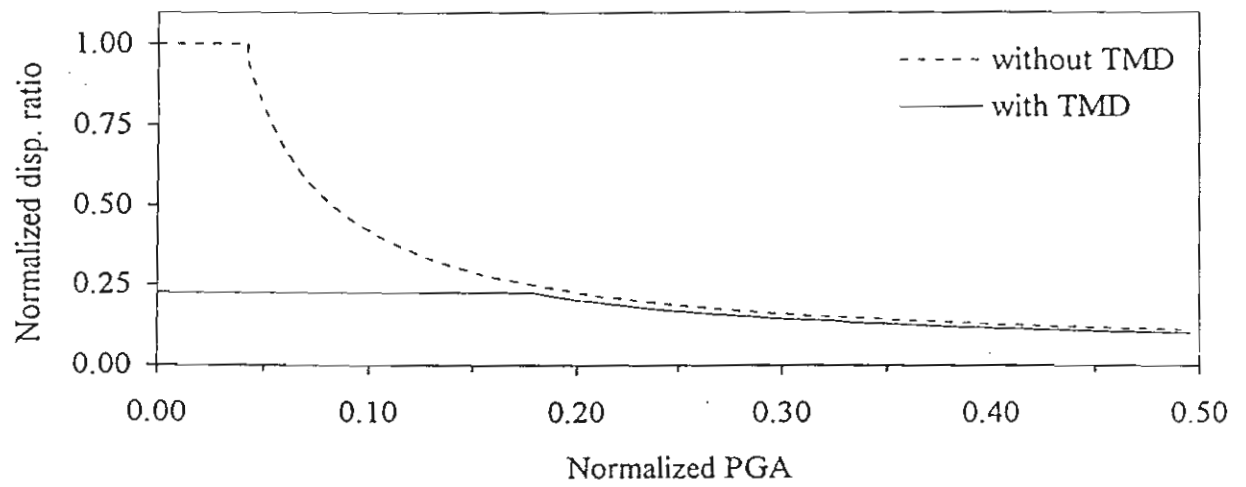


Figure 5 Normalized displacement ratio of structure against normalized PGA

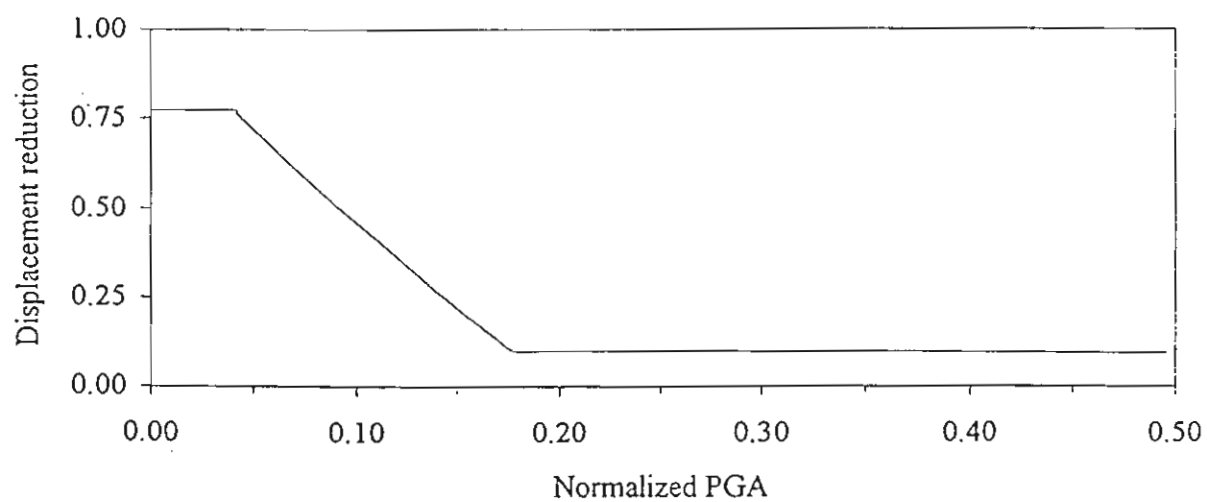


Figure 6 Displacement reduction of structure with TMD against normalized PGA

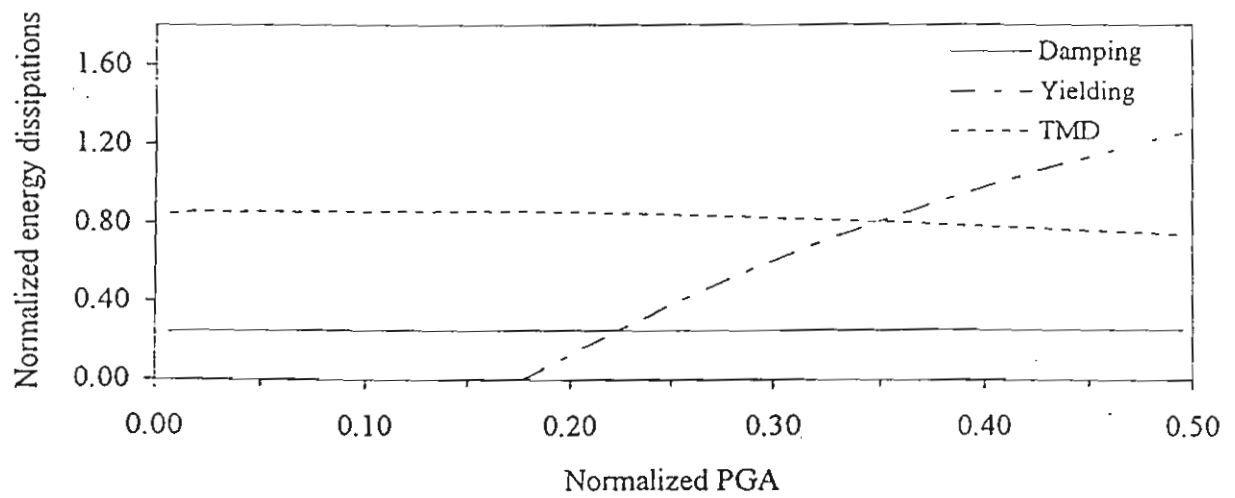


Figure 7 Normalized energy dissipation in structure with TMD against normalized PGA

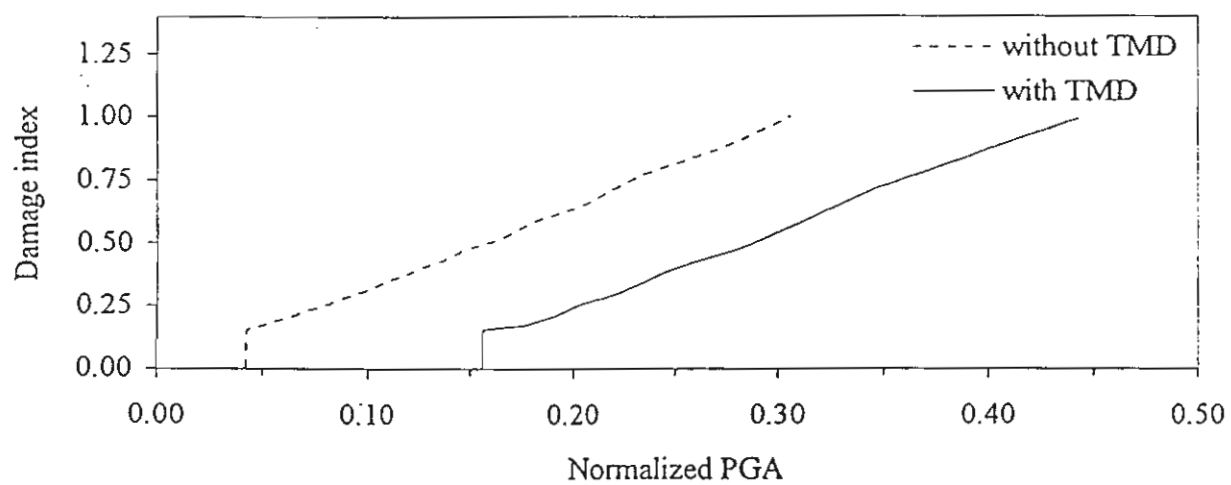


Figure 8 Damage index of structure against normalized PGA

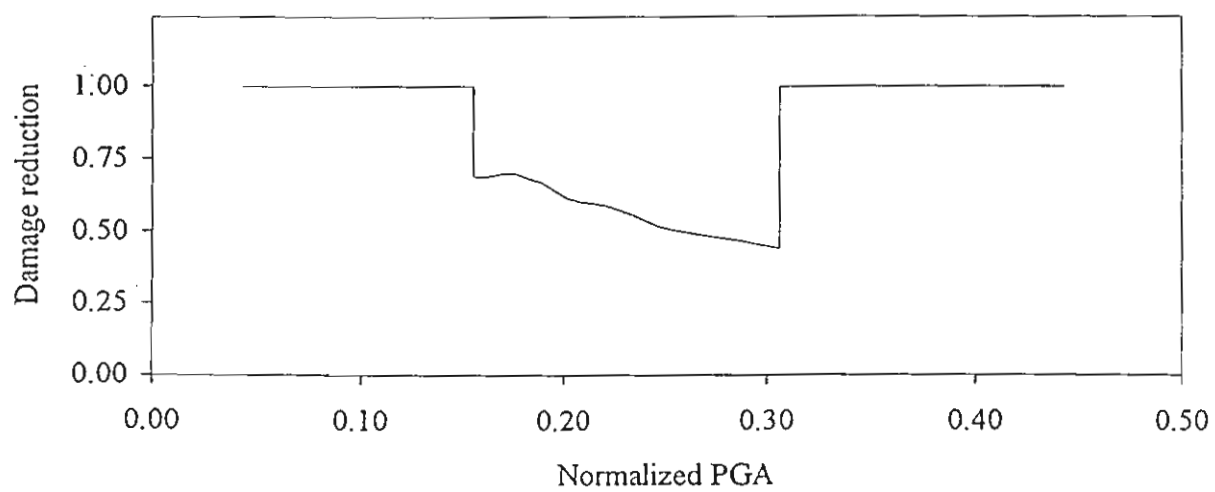


Figure 9 Damage reduction of structure with TMD against normalized PGA

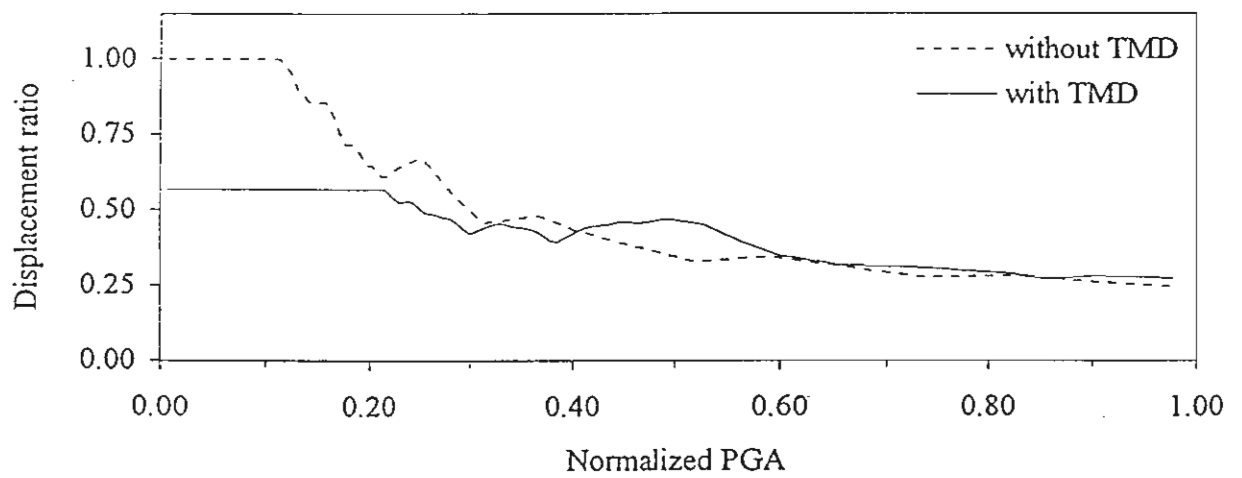


Figure 10 Displacement ratio of the structure against normalized PGA

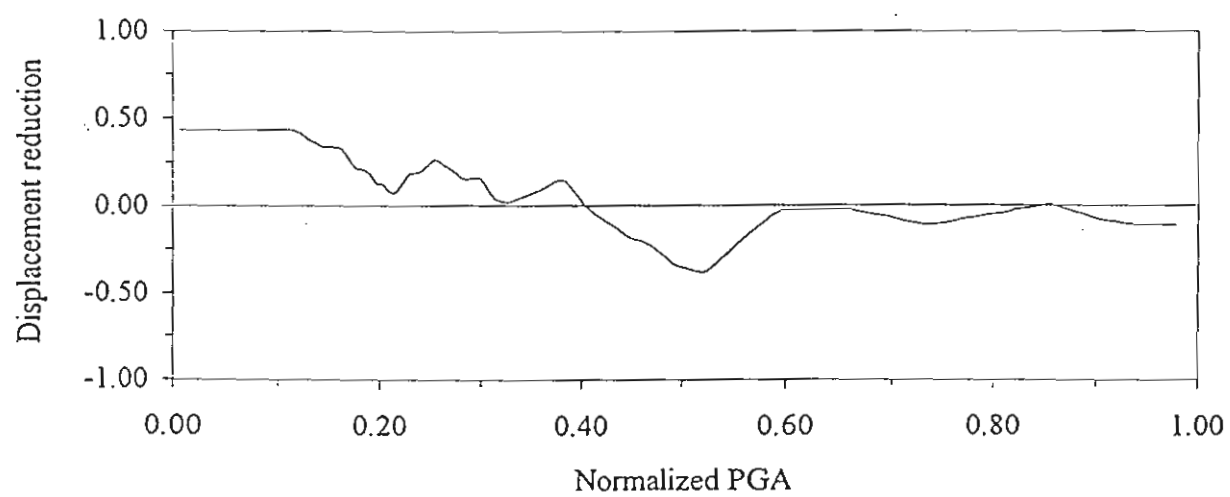


Figure 11 Displacement reduction of the structure with TMD against normalized PGA

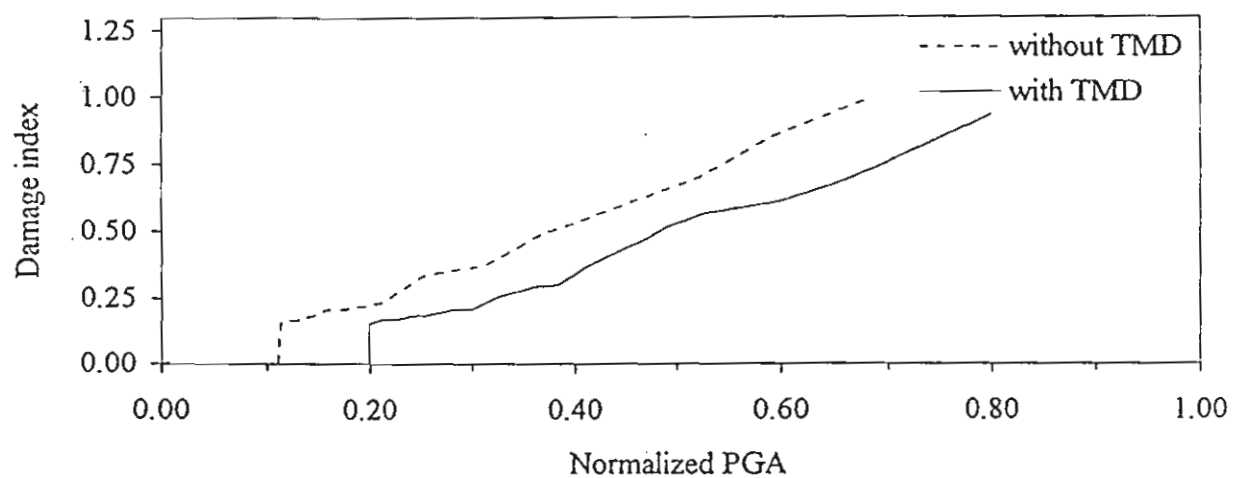


Figure 12 Damage index of structure against normalized PGA

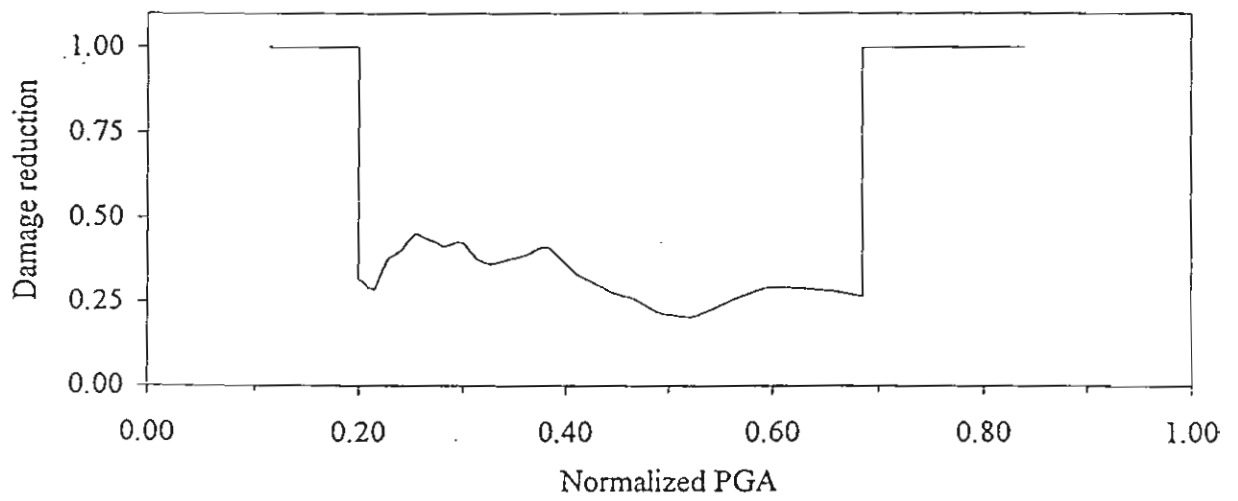


Figure 13 Damage reduction of structure with TMD against normalized PGA

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บทความที่ได้จัดเตรียมต้นฉบับ

- ก. Lukkunaprasit, P., and Sittipunt, C. "Ductility Enhancement of Moderately Confined Concrete Tied Columns with Hook-Clips" (submitted to ACI Structural Journal)
- ข. Lukkunaprasit, P., and Thepmangkorn, J. "Load History Effect on Cyclic Behavior of R.C. Tied Columns. (to be submitted to J. Struct. Engrg., ASCE)
- ค. Lukkunaprasit, P., and Wanitkorkul, A. "Retrofit of steel moment-resisting frames subjected to long-distance earthquakes considering weld fractures" (to be submitted to J. of Earthquake Engineering)

Ductility Enhancement of Moderately Confined Concrete Tied Columns with Hook-Clips

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ABSTRACT

An experimental investigation of the effectiveness of hook-clips in improving the performance of conventional 90-deg hook ties and ACI crossties in moderately confined reinforced concrete tied columns is described. The tie configurations provided in the five large-scale specimens tested included 90-deg hook ties and ACI crossties, with and without hook-clips, and 135-deg

hook ties. The columns were subjected to moderate levels of compression and cyclic lateral loads. The hook-clips were found to be effective in improving the performance of concrete columns confined with 90-deg hook ties and ACI crossties, resulting in the displacement ductility factor and energy dissipation capacity to be increased by about 85% and 400%, respectively.

Keywords: 90-deg hook; ACI crosstie; hook-clip; reinforced concrete column; ductility; energy dissipation.

INTRODUCTION

Although extensive studies of reinforced concrete (r.c.) columns confined with 135-deg hook ties have been carried out,¹⁻⁶ little work has focused on the performance of 90-deg hook ties, in spite of the fact that crossties with a 135-deg hook at one end and a 90-deg hook at the other (ACI crossties) are permitted by the ACI Code⁷, even in areas of high seismic risk. Razvi and Saatcioglu⁸ tested two specimens with 90-deg hook ties, and the results indicated that they were inferior to columns confined by 135-deg hooks at axial strains in excess of about 0.015. Sheikh and Yeh⁹ investigated the behavior of tied columns with different reinforcement and tie configurations under medium to high axial load levels and flexure. Crossties with 90-deg hooks were reported to cause brittle failure and to be harmful rather than beneficial, especially at high axial loads. Lynn et al.¹⁰ tested eight full-scale reinforced concrete columns having details widely used before the mid-1970's in the U.S.A. and including 90-deg hook tie details among others. Cyclic load-displacement curves were obtained for light and moderate level axial loads. The poor performance of 90-deg hook ties was evident, leading to rapid loss of gravity load resistance. Wehbe et al.¹¹ tested four r.c. tied

columns bound by 135-deg hook hoop ties and ACI crossties with moderate confinement. In all specimens, it was observed that opening of the 90-deg crosstie hooks initiated failure, leading to buckling of the outer longitudinal steel. Subsequently, the 135-deg hooks also started to open up.

The deficiency of 90-deg hook ties in columns was witnessed in past earthquakes in bridges, reinforced concrete buildings and steel reinforced concrete structures.¹²⁻¹⁴ Despite their poor performance, 90-deg hook ties are still used extensively worldwide in low to moderate seismic risk regions because of the ease of their placement compared with the 135-deg hooks. Ninety-degree hook ties are even more appealing in developing countries where laying of reinforcing bars is quite commonly not practiced to a high level of precision, making it extremely difficult to put 135-deg hook ties in place when the vertical bars are misaligned. Recently, Lukkunaprasit¹⁵ introduced a simple device called a "hook-clip" to be clipped onto the conventional 90-deg hook ties or crossties at the sites. Experimental tests on axially loaded short columns revealed that the performance of r.c. tied columns with 90-deg hooks and hook-clips was comparable to that of columns with 135-deg hook ties.

RESEARCH SIGNIFICANCE

In view of the importance of vertical load resistance members, it is essential to have ductility in columns to ensure vertical load resistance even in zones with low to moderate seismic risk. Enhancement of performance of 90-deg hook tied columns would contribute to reduced damage due to earthquakes in such seismic risk regions. Furthermore, while there exist numerous test data on r.c. tied columns with ductile detailing for areas of high seismicity, there is a paucity of test results for lower ductility

demand suitable for moderate seismic risk regions. The experimental results from this study would form a valuable addition to the database of r.c. tied columns under cyclic loading. The effectiveness of hook-clips in improving the performance of conventional 90-deg hook ties and ACI crossties in columns for a moderate level of ductility was investigated. Enhancement in displacement ductility and energy dissipation capacity was also examined.

THE HOOK-CLIP

To prevent premature opening of 90-deg hooks, a supplementary tie or "hook-clip" has been devised which is to be embedded in the concrete core with its hooks holding the legs of the hook ties. The clip resists opening of the 90-deg hook after loss of the concrete cover. Fig. 1(a) shows the details of the clip proposed for binding 9 mm diameter ties or smaller. The hook-clip may be employed to clip the legs of any hoop tie or crosstie with 90-deg hooks (see Fig. 1(b)). With the clips prefabricated, they can be applied easily at the site, without any welding.

EXPERIMENTAL PROGRAM

Test specimens

Five column specimens 400mm by 400mm in cross section and 1500mm in height served as test specimens. Each test unit was reinforced with 16 longitudinal deformed bars of 20mm (DB20) nominal diameter. Transverse reinforcement consisted of 9mm diameter hoop ties and ACI crossties, with consecutive crossties alternated end for end along the axis of the column. The ties were supplied with either 90-deg or 135-deg hooks, depending on specimens. Each hook had an inside radius of twice the tie diameter and an extension of 6 bar

diameters, but not less than 60mm. The reinforcement detailing was in accordance with the non-seismic detailing provisions in the ACI Code⁷. Fig. 2 depicts a typical column cross section, and Table 1 lists the relevant data of the test specimens.

It should be noted that a relatively large bar size (viz. 20mm diameter) was used for the longitudinal reinforcement so that when the bars buckled, a large outward thrust would be exerted on the ties, which would, in turn, try to pull the hook-clips out of the confined core. The tie spacing provided (120mm) was smaller than that stipulated by ACI Code for non-seismic detailing, which allows as much as 300mm for the specimens tested. The closer spacing was chosen in view of the higher demand on the ties in providing lateral restraint for the longitudinal bars when buckled in shorter unsupported lengths. Consequently, a higher demand was also imposed on the hook-clips in order to prevent the ties from opening. Nevertheless, as can be seen in Table 2, the lateral reinforcement provided, A_{sh} , was only 39%-52% of the minimum amount required by ACI in areas of high seismicity.

Special care was taken to achieve the following tolerances in construction: cross sectional dimensions ± 1 percent; column height ± 1 percent; tie dimensions in the critical region ± 1.2 percent; tie spacing in the critical region ± 3 percent; widths and lengths of hook-clips ± 3 percent; and verticality of specimen $\pm 1/500$.

Material properties

All specimens were made of normal-strength materials. Normal-weight concrete with a maximum aggregate size of 20mm was used. The concrete compressive strengths of the standard concrete cylinders on the day of testing, f_{cs} , were in the range of 30.5-38.9 MPa.

The reinforcing steel used consisted of deformed bars with an average yield strength, f_y , of 472 MPa for longitudinal reinforcement, and smooth round bars with yield strengths, f_{yh} , in the range of 297 to 318 MPa for transverse steel. The average modulus of elasticity of the reinforcing bars was 212,000 MPa. The clips were fabricated from 5mm diameter mild steel bars whose yield strength and modulus of elasticity were 450 MPa and 204,500 MPa, respectively.

It should be noted that, except for the slight variation in concrete strengths (about 15% from the mean value) and the hook configurations, all specimens were basically the same in physical properties. Specimens CF90/0.30, CF135/0.30 and CFL90/0.30 were designed to investigate the ductility performance of conventional 90-deg hooks, 135-deg hooks and 90-deg hooks with hook-clips, respectively. The label 0.30 designates an axial stress level of 0.30 f_{cs} (based on gross cross sectional area). The performance of hook-clips was re-confirmed with another set of specimens, CFL90/0.37 and CF135/0.37, which were compressed to a higher axial stress level of 0.37 f_{cs} .

Test setup

Fig. 3 shows the schematic diagram of the test setup. The column footing was tied down to a strong floor by six high-strength steel bars post-tensioned to a total force of 3000 kN. In addition, a strut-and-tie system was employed to further provide lateral restraint to the foundation to minimize its movement. The axial load on each specimen was applied by means of a hydraulic jack bearing against the column top and a load transfer girder sitting on top of the jack. The reaction from the loading jack was resisted by two $\varnothing 40\text{mm}$ high-strength steel bars which tied the transfer girder to the foundation. A calibrated 1000-kN hydraulic actuator was employed to supply the cyclic lateral force, which was applied through a shaft placed in an embedded sleeve near the top of the column. Although the specimens were set up with extra care to minimize the eccentricity of the lateral force to within 4mm on average, a lateral bearing system was also utilized to prevent any out-of-plane movement of the column during testing.

Instrumentation

Linear variable differential transformers (LVDTs) were employed to measure the lateral displacements of the column along its height. The second and third hoop ties above the base were instrumented with electrical resistance strain gages, placed at the locations shown in Fig. 2. For specimens CFL90/0.37 and CF135/0.37, additional strain gages were also attached to the crossties at the second, third and fourth levels, in the direction of loading. An angle measuring device was used to monitor the inclination of the column top.

The vertical load was measured by means of a calibrated pressure gage. The 1000 kN hydraulic actuator for horizontal load application was fitted with a load cell. Signals, from the load cell, LVDTs and strain gages were connected to a computerized data acquisition system.

Testing procedure

The test specimens were first subjected to preliminary loadings under 50 percent of the specified axial load and a very small lateral load (in the order of 30 kN) in order to determine accidental eccentricities of the loadings, as well as to assure proper functioning of all measurement devices. Any necessary corrective measures would then be applied to ensure that the accidental eccentricity in the vertical load was within a tolerance of one percent of the column width, on average.

Actual testing was carried out following the general procedure proposed by Watson and Park⁵. After the application of the specified axial load, the lateral force was load-controlled to ± 75 percent of the theoretical lateral yield value, H_v , computed on the basis of the ACI Code without any strength reduction. The experimental yield displacement, Δ_y , was then extrapolated from the average of the measured displacements at $+ 0.75 H_v$ and $- 0.75 H_v$ (Fig. 4). Subsequently, each specimen was subjected to displacement-controlled cyclic loading, starting from the displacement ductility level of 1. The displacement ductility level was incremented at an interval of 1, in general, with two cycles of loading performed for each ductility level until the ultimate capacity was reached. Failure was defined as the state when the capacity of the specimen during the loading cycle considered dropped by more

than 20 percent of the maximum capacity of the specimen. The associated displacement is denoted by Δ_u , and the displacement ductility factor is

$$\mu_{\Delta} = \Delta_u / \Delta_y \quad (1)$$

Loading was applied at a very slow rate, with one cycle completed in about one hour. The slow rate of loading permitted control of the constant axial force by manual operation of the hydraulic pump.

TEST RESULTS

Test observations

Specimen CF90/0.30 (with 90-deg hooks but without hook-clips) exhibited normal flexural and shear cracks when loaded through 2 cycles at ductility 1.0, and only a few small spalling cracks developed at the edges in the plastic hinge zone. However, during the first push cycle at ductility 2.0, widespread spalling cracks occurred, which were caused by the popping out of the 90-deg hooks of the hoop tie and crosstie in the second tie set above the footing. At the end of the second cycle at ductility 2.0, those cracks became excessive. During the next push cycle at ductility 3.0, a major part of the concrete cover in the plastic hinge zone on the compression face spalled off, exposing the 90-deg hooks. The member consequently lost its load carrying capacity. The buckling mode of the vertical bars was not clear at this stage. However, after being loaded through another half cycle, it could clearly be seen that the longitudinal bars had buckled over approximately twice the tie spacings, indicating the inadequacy of the 90-deg hooks in the critical hoop tie and ACI crosstie in restraining longitudinal bars at the tie position (Fig. 5).

Specimens CF135/0.30 (with 135-deg hooks) and CFL90/0.30 (with 90-deg hooks and hook-clips) behaved in a similar manner up to ductility level 2.0 with stable hysteresis loops and little strength degradation. In contrast to specimen CF90/0.30, which had already developed significant spalling cracks at this ductility level, only a few small ones occurred in CFL90/0.30 and CF135/0.30. When specimen CF135/0.30 was pushed through the first cycle of ductility level 3.0, the spalling cracks, which had developed earlier at ductility level 2 as small cracks along one edge near the base, rapidly propagated with increasing width and length. Spalling cracks and swelling of the concrete cover around the third tie set above the base also developed due to expansion of the hoop tie. During the last cycle at ductility 3.0, more spalling near the base occurred, and swelling of the concrete in the vicinity of the 90-deg end of the ACI crosstie in the second tie set was evident. The next (incomplete) push cycle to ductility level 4.0 saw excessive spalling and swelling of the concrete covering between the first and third tie sets, with opening of the 90-deg hook in the ACI crossties and eventual buckling of the longitudinal bars (Fig. 6). It was also observed that the 135-deg hook of the most severely stressed hoop tie was so deformed that it opened up substantially, indicating deficient anchorage of the hook due to the short extension leg provided for non-seismic design.

Specimen CFL90/0.30 exhibited remarkable behavior. Up to the second cycle, at ductility level 3.0, the overall appearance of the column was still in fairly good condition except for minor surface spalling of the cover at the base on the compression faces (which had occurred since ductility level 2), and some wide vertical spalling cracks near the edges in the plastic hinge zone. Swelling of the concrete cover on the compression face was observed near the location of one 90-deg end of the ACI crosstie at the second tie set above the base, indicating expansion action of the ties and hooks. However,

reinforcing bars were not exposed until the specimen was loaded to ductility level 4, when extensive spalling of the concrete cover took place. Bending of an inner longitudinal bar due to buckling could also be clearly observed at this stage (Fig. 7). Of particular significance is the integrity of the hook-clips in holding the legs of the 90-deg hook tie even at a substantial drift of 4% as witnessed in Fig. 8. It was remarkable that the longitudinal bars possessed significant post-buckling strength and the ties and hook-clips were resilient in confining the core, so that a significant amount of the peak load could be sustained, without abrupt failure, through one full cycle before eventual failure by total buckling of the longitudinal bars. The buckling shape of the longitudinal bars in the plastic hinge zone resembled that of specimen CF135/0.30 (Fig. 9).

Specimen CF135/0.37 displayed extensive spalling cracks near the edges in the plastic hinge zone when loaded to ductility level 2. Swelling of the concrete cover was significant along the third hoop level, but less so at the second hoop level. The edges spalled off during the first cycle of $\mu_d = 3$, but the longitudinal bars were still not visible. Just before the completion of the second cycle at ductility level 3, there was a drastic drop in lateral load resistance, followed by rapid widening of a major shear crack which had previously been minute, leading to eventual failure.

The overall appearance of CFL90/0.37, on the other hand, was much better than that of its counterpart, CF135/0.37. Similar crack patterns were observed in general, but the extent of cracking and damage was significantly less in the former than in the latter. At $\mu_d = 2$, only a few minor spalling cracks occurred over small areas at the base and at the third tie level. The cracks developed into significant ones at the third hoop tie level at ductility factor 3, with a clearly noticeable drop in lateral load

resistance. This prompted a reduction of the displacement increment to $0.5\Delta_y$ for the next (and last) loading cycle. Buckling of vertical bars in the plastic hinge zone led to an excessive drop in capacity and termination of the test after $1\frac{1}{2}$ cycles of loading at ductility level 3.5.

Lateral load-displacement hysteretic response

The lateral load-displacement hysteretic responses for the test specimens are shown in Fig. 10a - 10b and 11. The curves clearly indicate flexural dominated characteristics. It is interesting to note that specimens CF135/0.30 and CFL90/0.30 experienced stable hysteresis loops up to ductility level 3.0, with similar general characteristics. While the specimen with 135-deg hooks and (unclipped) ACI crossties suffered sharp decrease in lateral load resistance during the next (and last) loading cycle to ductility level 4.0, the specimen with hook-clips exhibited a stable hysteresis loop in the same cycle, with little strength degradation. The latter was able to sustain 2 complete cycles at this ductility level, although significant degradation in strength and stiffness occurred in the second cycle at $\mu_d = 4.0$. The actual ductility factor that could have been attained by this specimen was estimated to be 3.7 which was based on equivalent energy dissipation and the condition that the loss in strength not to exceed 20 percent.

The sudden drop in lateral load resistance was even more pronounced in specimen CF90/0.30 when it was being pushed to $3\Delta_y$. The rapid decrease in load resistance was caused by the opening of the 90-deg hook of the ACI crosstie in the second tie set above the base, when the longitudinal bars buckled over approximately two tie spacings.

Specimen CFL90/0.37 exhibited a more stable hysteretic response than CF135/0.37, with much less strength degradation, when loaded from the second cycle at ductility 2.0 to the second loading cycle at $\mu_A = 3$. Reduction of the peak load at ductility factor 3 was 22 percent for specimen CF135/0.37, compared with only 8 percent for specimen CFL90/0.37. However, prior to unloading from the second cycle at $\mu_A = 3$, specimen CF135/0.37 rapidly lost its load capacity (by more than 45 percent) due to buckling of the vertical bars at a displacement of $+2.54\Delta_y$. The specimen with hook-clips, on the other hand, could still carry 85 percent of the peak load, and even sustained one complete cycle at $\mu_A = 3.5$ with a remarkable sustained capacity before final failure.

Ductility performance

The displacement ductility factors attained by specimens CF90/0.30, CF135/0.30 and CFL90/0.30, under an axial stress level of $0.3 f_{ca}$, were 2, 3 and 3.7, respectively (see Table 3). It may be noted that the displacement ductility of CFL90/0.30 closely agreed with the value of 3.8 predicted by the formula suggested by Wehbe, et al.¹¹ for members with seismic detailing, indicating the effectiveness of the hook-clips in enhancing ductility performance of columns with 90-deg hook ties. The ductility performance of specimens without clips was expected to be unsatisfactory due to the use of non-seismic detailing, and hence it was not compared with that predicted by the Wehbe et al. equation.

The effectiveness of the hook-clips was again confirmed by specimen CFL90/0.37, which was able to sustain the same displacement ductility factor

as specimen CF135/0.30 even though it was subjected to a higher level of axial load.

Strains in transverse steel

Focus was first placed on measurement of the hoop tie strains in the plastic hinge zones in specimens CF90/0.30, CFL90/0.30 and CF135/0.30. It was unfortunate that some strain gages got damaged (some caused by the spalling concrete) which resulted in incomplete data so that comparison of the maximum axial strains in the hoop ties could not be made. Furthermore it was found that the strains in the hoop ties were much influenced by bending of the ties caused by the lateral pressure exerted by the core and/ or buckling of the longitudinal bars. In the last two specimens, therefore, attention was paid to the measurement of the strains in the crossties which were predominantly in tension. It was found that the most severely strained crosstie in specimen CFL90/0.37 developed a maximum strain of 1.5 times that in CF135/0.37. In fact, the former attained a strain of 0.0015, slightly higher than the yield value of 0.0014. Better anchorage of the 90-deg hook ends with hook-clips after spalling of the concrete cover obviously resulted from effective restraint of the hook ends by the hook-clips, leading to the ability to develop higher strains in the crossties, and enhanced confinement of the concrete core.

Energy dissipation capacity

The ability of structures to withstand cyclic loading is commonly measured in terms of the energy dissipation capacity, which is defined as the summation of the energy E_i dissipated within each cycle i . The normalized energy dissipation capacity, E_N , is

$$E_N = \left(\sum_{i=1}^n E_i \right) / (H_{max} \Delta_y)$$

in which n is the number of cycles to failure, and H_{max} is the peak lateral load during cyclic loading.

The normalized energy dissipation capacities of the specimens tested are tabulated in Table 3 and the cumulative normalized energy capacities versus loading cycles are plotted in Fig. 12. Those cycles which resulted in a drop in lateral load resistance of more than 20 percent of the peak lateral load were excluded in the computation. It is remarkable that the energy dissipation capacity of specimen CFL90/0.30 was larger than the unclipped specimens CF90/0.30 and CF135/0.30 by about 400 and 90 percent, respectively. At a higher axial load level of $0.37 f_{ca} A_g$, the increase in the dissipated energy was less. At any rate, the energy dissipation capacity of CFL90/0.37 was still 1.5 times that of CF135/0.37.

Effectiveness of hook-clips

From the test results, together with the following observations, it is evident that the hook-clips were effective in improving the performance of the 90-deg hooks in the hoop ties and ACI crossties:

a) At each ductility level, the number and extent of spalling cracks were significantly lower in the specimens with hook-clips than in those without, indicating less pop-out action of the 90-deg ends of the ACI crossties and 90-deg hooks in the hoop ties due to containment by the hook-clips.

b) In contrast to CF90/0.30 with conventional 90-deg hooks, the clips in the CFL90/0.30 specimen were able to prevent premature opening of the 90-deg

hooks, leading to effective restraint of the vertical bars at the tie positions, and resulting in the vertical bars buckling by about half the buckling length of those without clips, when buckling commenced, as depicted in Fig. 7. The shorter buckling length allowed the specimen to sustain a higher load at the same deformation.

c) The ductility performance of the specimen with 90-deg hook ties and hook-clips far excelled that of the specimen without hook-clips. In fact, the former even performed better than the CF135/0.30, which was confined with 135-deg ties and ACI crossties (without hook-clips). The reason is that the ACI crossties with hook-clips in specimen CFL90/0.30 were effectively restrained from opening up by the hook-clips, in contrast to the unclipped ACI crossties in specimen CF135/0.30 which popped out at an earlier stage, leading to loss of structural integrity.

d) The crossties with hook-clips were able to develop larger strains than those without clips as mentioned earlier.

CONCLUSIONS

The hook-clips were found to be effective in improving the performance of reinforced concrete columns confined with 90-deg hook ties and ACI crossties, as evidenced by the significant enhancement of the ductility and energy dissipation capacity and the fact that, for the tie configurations studied, the buckling lengths of the longitudinal bars in specimens with hook-clips were about half of those in specimens without them when bar buckling commenced. In fact, for the specimens tested, the overall performance of the specimens with hook-clips under a moderate ductility demand was even superior to that of columns confined with 135-deg hook ties and conventional ACI crossties. The effective restraint of the 90-deg ends of crossties by hook-clips enabled the most severely strained crossties in the plastic hinge

region to develop a strain slightly higher than the yield value. The effectiveness of the hook-clips should be beneficial, even in regions of high seismicity, pending further investigation.

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CONVERSION FACTORS

1 MPa	=	145 psi
1 mm	=	0.0394 in.
1 kN	=	0.2248 kips

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Table 1 - Details of test specimens

Specimen	Concret strength f_{cs} MPa	Dimensions			Longitudinal reinforcement		Transverse reinforcement				Hook configuration	$P/f_{cs}A_g$
		Width, mm	Depth, mm	Height, mm	ρ_l %	f_y MPa	Diameter, mm	s , mm	f_{yh} , MPa	A_{sh}/sh_c %		
CF 90 / 0.30	38.9	398	397	1500	3.14	471	9	120	305	0.453	90° + ACI crossties; no clips	0.30
CF 135 / 0.30	35.7	398	396	1490	3.14	471	9	120	305	0.453	135° + ACI crossties; no clips	0.30
CFL 90 / 0.30	31.7	398	398	1500	3.14	471	9	120	306	0.453	90° + ACI crossties with clips	0.30
CF 135 / 0.37	30.5	399	397	1500	3.14	475	9	120	318	0.453	135° + ACI crossties; no clips	0.37
CFL 90 / 0.37	32.4	398	397	1500	3.14	471	9	120	297	0.453	90° + ACI crossties with clips	0.37

Note: ρ_l = longitudinal reinforcement ratio; s = center-to-center spacing between sets of ties; h_c = cross-sectional dimension of column core measured center-to-center of confining reinforcement; P = axial load; and A_g = gross area of column section.

Table 2 - Lateral reinforcement ratio in comparison with ACI Code⁷
(seismic design)

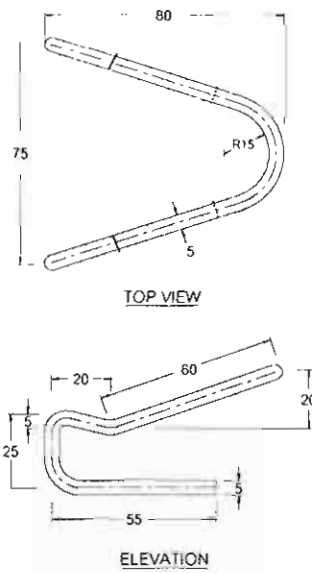
Specimen	$P/f_{cs}A_g$	Lateral reinforcement ratio $A_{sh}/(sh_c)$		
		Provided	ACI Code	$A_{sh}/A_{sh,ACI}$
CF 90 / 0.30	0.30	0.0045	0.0115	0.39
CF 135 / 0.30	0.30	0.0045	0.0105	0.43
CFL 90 / 0.30	0.30	0.0045	0.0093	0.49
CF 135 / 0.37	0.37	0.0045	0.0086	0.52
CFL 90 / 0.37	0.37	0.0045	0.0098	0.46

Note: $A_{sh,ACI}$ = minimum total cross-sectional area of rectangular hoops and crossties as specified by the ACI Code.⁷

Table 3 - Test results

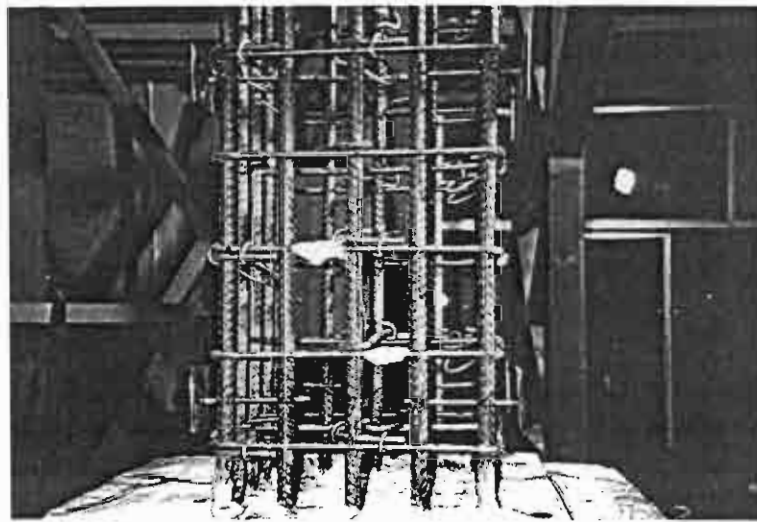
Specimen	Δ_y , mm	Δ_u , mm	$\mu_{\Delta,exp}$	$\mu_{\Delta,cal}$	H_{max} , kN	E_N , kN-mm	$E_N/(H_{max}\Delta_y)$	Failure mode
CF 90 / 0.30	14.3	28.4	2.0	-	315	16908	3.8	Flexure
CF 135 / 0.30	14.2	42.6	3.0	-	314	44806	10.1	Flexure
CFL 90 / 0.30	15.0	55.5	3.7	3.8	284	83032	19.3	Flexure
CF 135 / 0.37	14.2	42.4	2.5	-	295	27821	6.6	Flexure
CFL 90 / 0.37	13.2	39.6	3.0	3.5	303	41448	10.4	Flexure

Note: $\mu_{\Delta,exp}$ = displacement ductility factor from experiment; and $\mu_{\Delta,cal}$ = calculated displacement ductility factor in accordance with the formula proposed by Wehbe et al.¹¹



(Dimensions in mm)

(a)



(b)

Fig.1-(a) Details of hook-clip; and (b) Hook-clips engaging 90° hoop ties and crossties

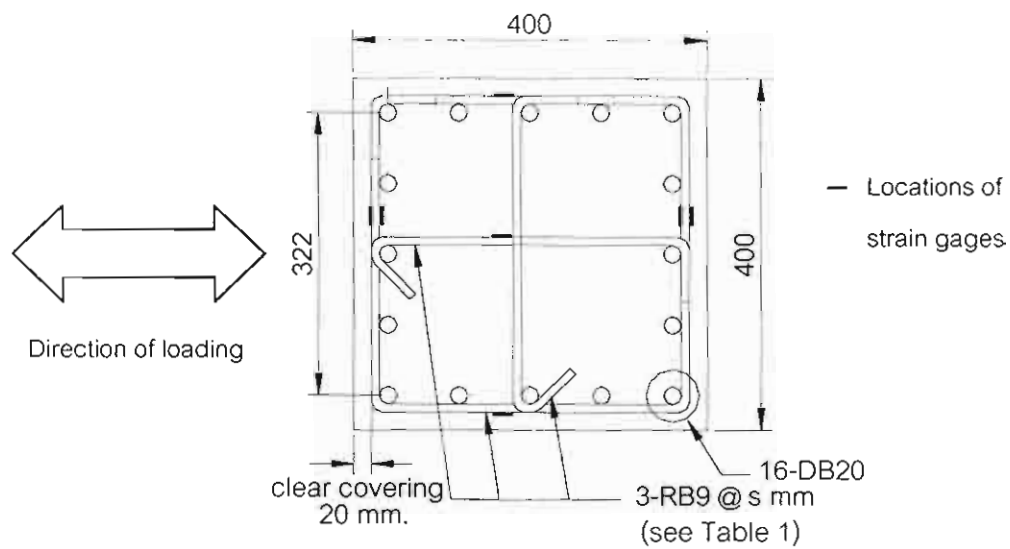


Fig.2-Column reinforcement detail

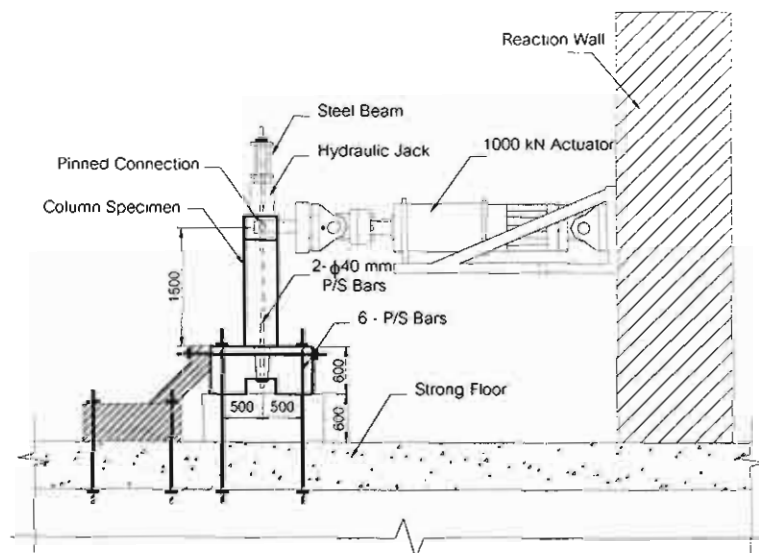


Fig.3-Test setup

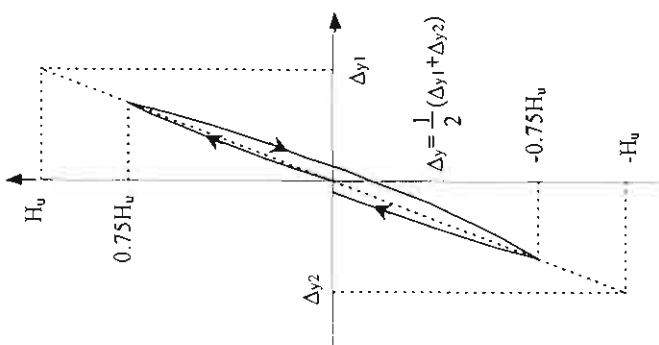


Fig. 4-Definition of first-yield displacement
(after Watson and Park⁵)

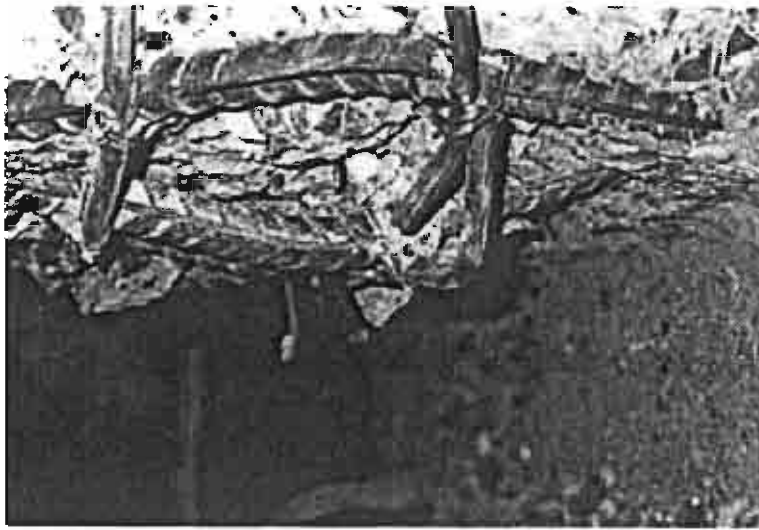


Fig. 5-Failure mode of CF90/0.30

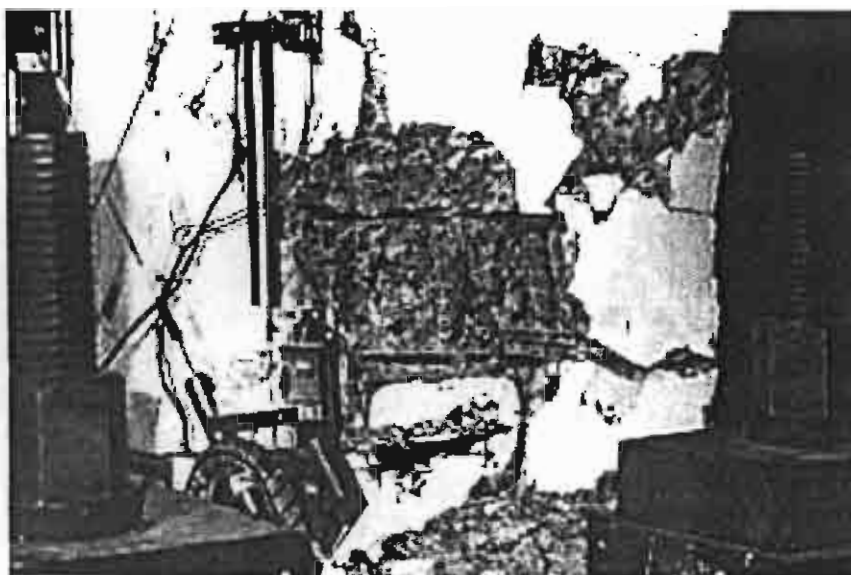


Fig. 6 - Failure mode of CF135/0.30

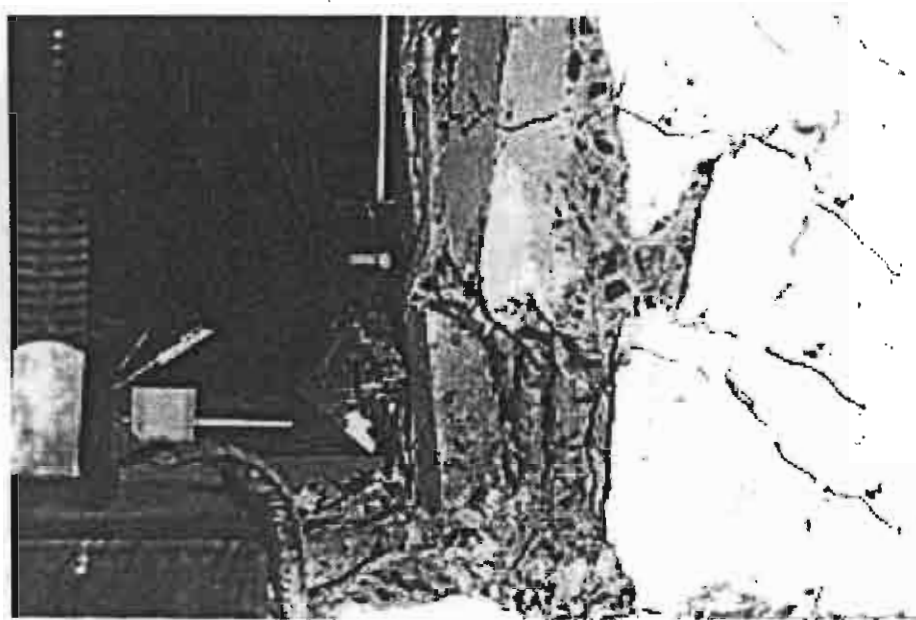


Fig. 7 - CFL90/0.30-Buckling of vertical bar in single tie spacing

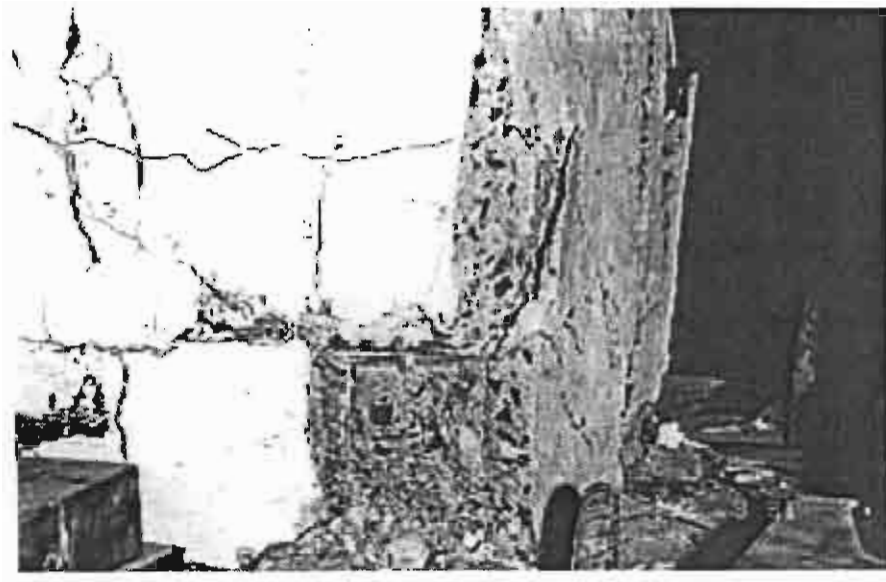
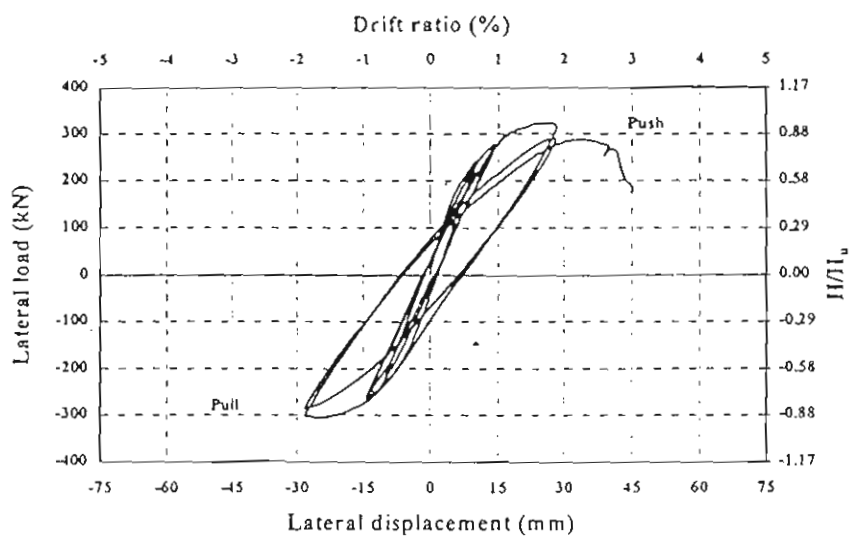


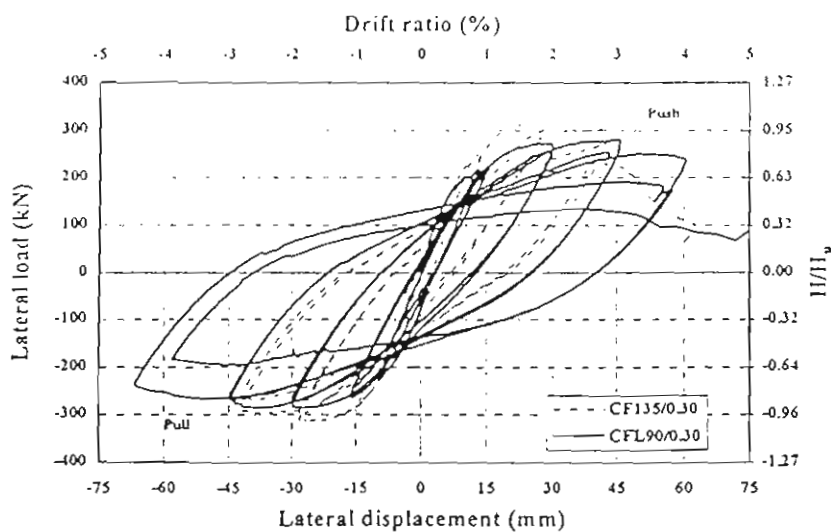
Fig. 8 - CFL90/0.30-Effective restraining of 90-deg hook legs by hook-clips at a large drift of 4%



Fig. 9 - CFL90/0.30-Effectiveness of hook-clips in restraining hook opening at failure



(a)



(b)

Fig.10 - Lateral load-displacement hysteresis: (a)CF90/0.30;
(b)CF135/0.30 (dashed line) and CFL90/0.30 (solid line)

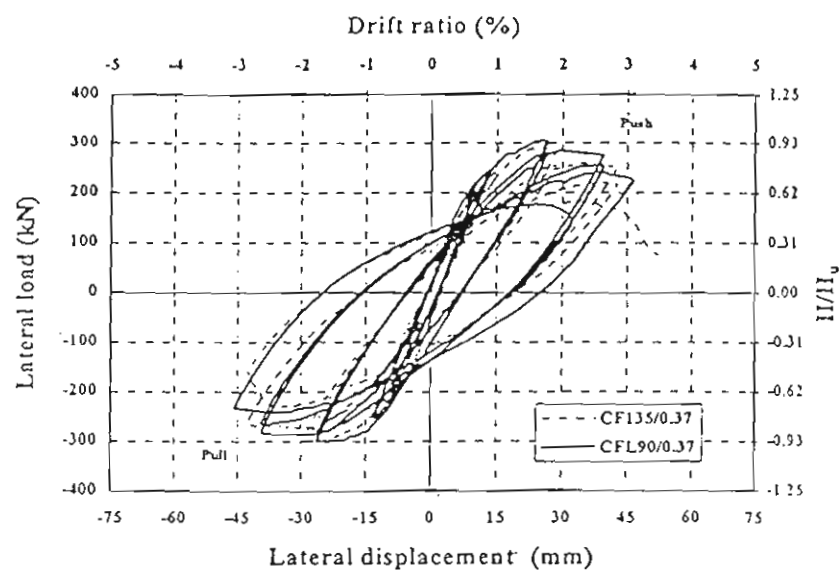
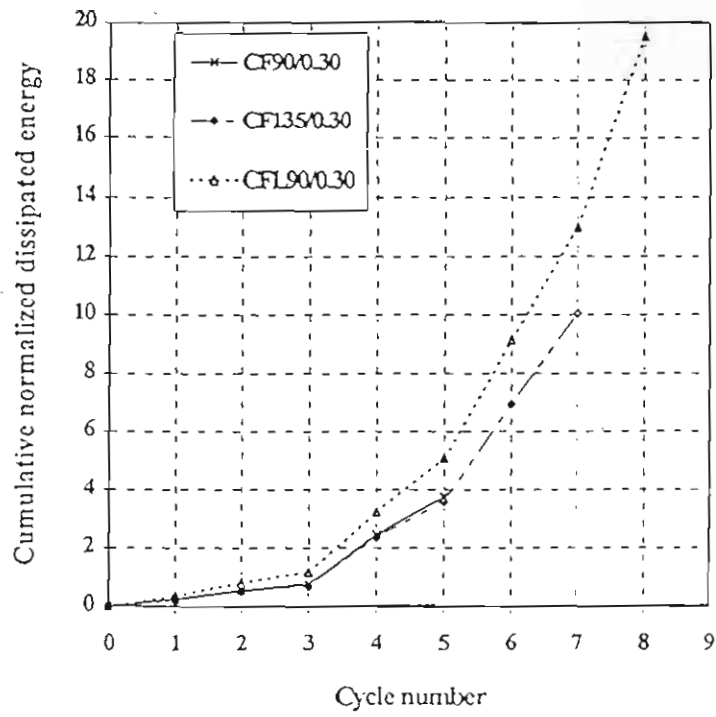
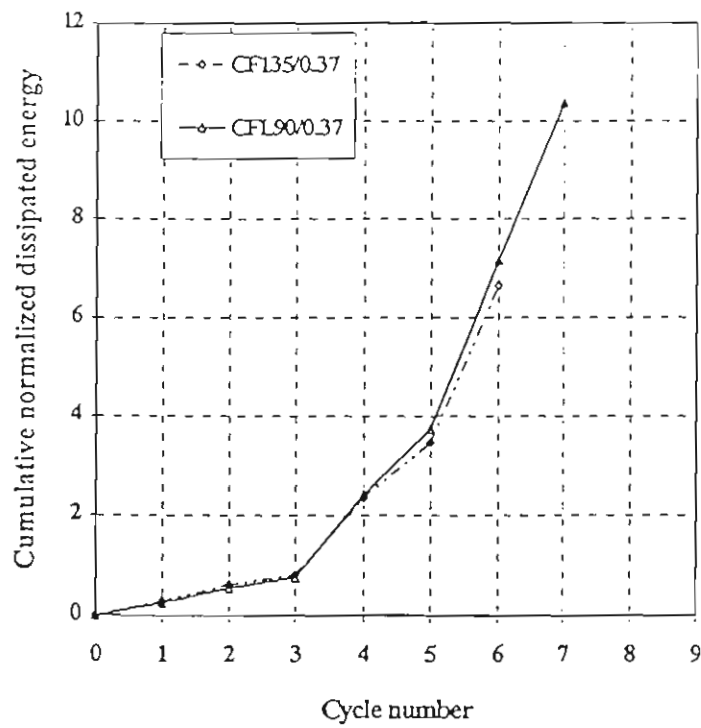


Fig. 11 - Lateral load-displacement hysteresis for specimens
CF135/0.37 (dashed line) and CFL90/0.37 (solid line)



(a)



(b)

Fig.12-Cumulative normalized dissipated energy: (a) Specimens CF90/0.30, CF135/0.30, and CFL90/0.30; and (b) Specimens CF135/0.37 and CFL90/0.37.

List of Notations

A_g	=	gross area of column section
A_{sh}	=	total cross-sectional area of transverse reinforcement
$A_{sh,ACI}$	=	minimum total cross-section area of rectangular hoops and crossties as specified by the ACI Code
E_i	=	dissipated energy within cycle i
E_h	=	normalized dissipation energy capacity
F_{cd}	=	compressive strength on day of testing of the standard concrete cylinder
f_y	=	yield strength of longitudinal reinforcement steel
f_{yh}	=	yield strength of transverse reinforcement steel
H_u	=	theoretical lateral yield load
H_{max}	=	peak lateral load during cyclic loading
h_c	=	cross-sectional dimension of column core measured center-to-center of confining reinforcement
P	=	axial load
s	=	center-to-center spacing between sets of ties measured along the axis of the column
Δ_u	=	ultimate tip displacement of column
Δ_y	=	yield displacement of column
μ_d	=	displacement ductility factor
$\mu_{d,cal}$	=	calculated displacement ductility factor in accordance with the formula proposed by Wehbe et al. ¹¹
$\mu_{d,exp}$	=	displacement ductility factor from experiment
ρ_l	=	longitudinal reinforcement ratio

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Load History Effect on Cyclic Behavior of R.C. Tied Columns

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Abstract : The effect of two commonly used load histories on the ductility and energy dissipation capacity of reinforced concrete tied columns was investigated. Based on regression analyses of the test results reported in the literature, the effective confinement reinforcement ratio was found to provide a good correlation with the performance indexes studied. The parameters accounted for include the material strength ratio, the amount of the transverse reinforcement, the axial load level and the effectiveness of the tie configurations. For moderate values of the effective confinement steel ratio, load history with increments of two times the yield displacement was found to be slightly more severe than that with one yield displacement increments. The effect of load history tends to be less significant for higher values of the effective confinement steel ratio.

INTRODUCTION

The influence of load history on the cyclic behavior of reinforced concrete (r.c.) members has been recognized for some time. Gosain et al. (1977) proposed the modified work index, I'_w , as a measure for evaluating the severity of the loading for r.c. members. They also pointed out limitations of the method when applied to significantly different load histories. For instance, loading to a deflection of 10 times yield displacement twice would likely yield more severe damage than loading four times to a deflection of 5 times the yield value, even though both loadings yield approximately the same value of work index. Noting the weak correlation of I'_w with the test data, Darwin and Nmai (1986) proposed an energy dissipation index, D_i , designed to provide an objective measure of beam response. It was found that D_i was strongly controlled by the maximum shear stress, ν_m , the concrete strength, f'_c , and the transverse steel capacity, ν_s , in the form

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of the parameter $(v_s f'_c)^{0.5} / v_m^{1.5}$. Hwang and Scribner (1984) carried out experimental investigations on the effect of load history on reinforced concrete cantilever beams. The sequence of application of large and small deformations was found to have relatively small effect on the strength and stiffness degradation, and total energy dissipation capacity of the member. The most significant factor was the maximum displacement the members experienced. A limited investigation by Wight and Sozen (1975) seemed to suggest the same result.

It may be noted that most of the studies mentioned dealt with beams, with relatively little work carried out on columns. In the majority of studies on the cyclic behavior of reinforced concrete tied columns, the material properties, the amount of confinement steel and the effect of axial load level, are normally taken as the main parameters (e.g. Soesianawati et al., 1986; Azizinamini et al., 1992; Watson et al., 1994; Wehbe et al., 1999). The effect of tie configuration effectiveness has recently been considered by Sheikh and Khoury (1997). However, no single standard load history has been employed, and it is difficult to compare the performance of r.c. members tested by different investigators using different load histories. Furthermore, constitutive relationships have been normally assumed to be independent of load histories. It was thus the primary objective of this study to investigate the effect of load histories on the ductility and energy indexes of r.c. tied columns under cyclic loading. A parameter which influences the cyclic performance of r.c. tied columns was also identified.

PERFORMANCE EVALUATION

To evaluate the performance of columns on a rational basis, it is theoretically preferable to resort to a common and systematic approach in determining the various response parameters such as the yield displacement, the ultimate displacement, etc. Following the procedures outlined by Sheikh and Khoury (1993), the envelope curve (which results from the average of values in both directions) to hysteresis loop is first constructed. The displacement ductility factor, μ_Δ , is defined as

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_y} \quad (1)$$

where Δ_u is the displacement reached when the capacity of the specimen dropped to 80% of the peak value, which is considered as the failure load, and Δ_y is taken as the displacement value attained at the peak lateral load, H_{max} , assuming an initial stiffness K_i as shown in Fig. 1.

In computation, the initial stiffness may be difficult to obtain accurately due to some fluctuations in displacement data at small loads. Assuming that the initial response is essentially elastic at a lateral load not exceeding 30 percent of the maximum capacity of the specimen, the secant modulus at this load level was taken as a good approximation of the initial stiffness for practical purposes.

The dissipated energy, E , is defined as the cumulative energy dissipated within each cycle i , shown as the hatched area E_i in Fig. 1. Thus

$$E = \sum_{i=1}^n E_i \quad (2)$$

in which n is the number of cycles to failure. The normalized dissipated energy, E_N , is

$$E_N = \frac{1}{H_{max} \Delta_y} \sum_{i=1}^n E_i \quad (3)$$

Rather than computing the dissipated energy, Gosain et al. (1977) proposed the following work index as an alternative performance indicator

$$I_w = \sum_{i=1}^n [(H_i \Delta_i) / (H_{max} \Delta_y)] \quad (4)$$

where H_i and Δ_i are the maximum values of the lateral load and displacement in cycle i , respectively, averaged in both directions. Eq. (4) was further simplified to

$$N_{\Delta} = \sum_{i=1}^n (\Delta_i / \Delta_y) \quad (5)$$

which can be interpreted as the cumulative ductility ratio (Sheikh and Khoury, 1993).

Effective confinement steel ratio

It has been pointed out that while the ACI Code provisions for the design of confinement steel for high seismic risk regions are conservative for low axial loads, they may not be sufficient for high levels of axial load. Watson and Park (1994) proposed a design equation which accounts for the ductility performance in terms of curvature ductility factor, μ_{ϕ} , the influence of axial load level as well as material and dimensional parameters. Sheikh and Khoury (1997) developed a procedure for the design of confinement steel considering axial load level, tie configuration, and the expected curvature ductility performance.

Based on the concept of Sheikh and Khoury, it is suggested that the effective confinement steel, $A_{sh}(k'_e)$, for a targeted ductility demand, μ_{Δ} , be given by the ACI confinement steel, $A_{sh,ACI}$, modified by the axial load and ductility level functions, i.e.

$$A_{sh}(k'_e) = A_{sh,ACI} g(\mu_{\Delta}) Y_p \quad (6)$$

where k'_e is the confinement effectiveness coefficient normalized with respect to 0.75, which is the upper bound value assumed by the ACI Code for the efficiency of rectilinear ties in comparison with that of closely-spaced spirals; $g(\mu_{\Delta})$ is a nonlinear function of the displacement ductility demand; and Y_p is the axial load parameter given by (Sheikh and Khoury, 1997)

$$Y_p = 1 + 13(P/P_0)^5 \quad (7)$$

in which P is the axial load, and P_0 is the ultimate load capacity.

Several researchers have proposed different formulae to account for the non-uniformity in confining pressure resulting from bending of ties under lateral pressure, which causes reduction in confinement effectiveness. The simple empirical formula proposed by Saatcioglu (1996) for the confinement effectiveness coefficient, k_e , was used in this study:

$$k_e = 0.15 \sqrt{\left(\frac{b_c}{s_t}\right)\left(\frac{b_c}{s_l}\right)} \leq 1 \quad (8)$$

where b_c is the confined core dimension, center-to-center, of the perimeter hoop; s_t is the spacing of ties; and s_l is the spacing of the laterally supported longitudinal reinforcement.

Eq. (6) can be rewritten as

$$\mu_A = f(\rho_A) \quad (9)$$

where $f(\rho_A)$ is a nonlinear function of the effective confinement steel ratio to be obtained from a regression analysis of test results. The ‘effective confinement reinforcement ratio’, ρ_A , is defined as

$$\rho_A = \frac{A_{sh} k'_e}{A_{sh,ACI} Y_p} \quad (10)$$

This important parameter enables comparison of performance of column specimens with different transverse reinforcements (including tie arrangements) and different axial load levels.

Test Samples

Based on the number of test results available in the literature which needs to be large enough for a regression analysis, and the availability of digitized hysteresis loops, the number of test specimens that could be used in this study was only 17. The test samples can be broadly grouped into two categories in accordance with displacement histories (Fig. 2), viz.,

History type 1 : increments of $1\Delta_y$ with basically two cycles repeated for each displacement increment

History type 2 : increments of $2\Delta_y$ with basically two cycles repeated for each displacement increment

Histories types 1(a) and 2(a) are the basic ones. The others differ from the basic displacement histories in minor details. The extra third cycle at $6\Delta_y$ for specimen B2 with History Type 1(b) was accompanied by a much reduction in the lateral load capacity, and hence did not contribute to the energy computation. History Types 2(b) and 2(c) include extra cycles at $1\Delta_y$ which contributed less than 1% of the total energy. Judging from the stable hysteresis loops, the first two cycles at ductility level $7\Delta_y$ for specimen B1 with History Type 2(c) were approximated as equivalent to one cycle of $8\Delta_y$, assuming that one complete cycle at ductility level $8\Delta_y$ could have been attained if the loading history had strictly followed the $2\Delta_y$ increments.

Table 1 summarizes the details of test specimens whose results are used in the analyses. The main parameters, besides the displacement histories, include :

- a) material strength ratio f'_c / f_{yh} in the range of 0.060-0.136, where f'_c is the concrete compressive strength and f_{yh} is the yield strength of the transverse steel;
- b) shear span-to-depth ratio : 3.0 or larger;
- c) longitudinal reinforcement ratio : 1.51-2.45%;
- d) transverse steel in the range of 29% - 236% of that specified in ACI code (1999);
- e) axial load level of 0.08–0.77 of the concrete gross section capacity;
- f) tie configurations as shown in Fig. 3.

All column specimens were made of normal strength concrete with conventional hoops and/ or ties, and all exhibited flexure mode of failure. The data on hysteresis loops were obtained from the web site: www.ce.washington.edu/~peera1.

ANALYSIS OF RESULTS

Displacement ductility factor

The calculated values of the displacement ductility factor, $\mu_{\Delta,cal}$, are tabulated in Table 2 and plotted in Fig. 4 as functions of the effective confinement reinforcement ratio, ρ_A . The values of $\mu_{\Delta,exp}$ reported (or implied) by the researchers are also included in Table 2. It may be observed that for load history Type 1, the discrepancy in $\mu_{\Delta,cal}$ and $\mu_{\Delta,exp}$ is in the range of 5.8-20.0%, which is acceptable. However, for load history Type 2, the agreement became worse for some specimens. For instance, for specimen UNIT2 by Soesianawati et al. (1986), the test result suggests a value of $\mu_{\Delta,exp}$ of 6.0, much smaller than the computed value of 9.0. To be capable of reaching a ductility factor of 9.0, the specimen would have had to be able to sustain 10 (main) cycles (with adequate capacity) rather than 6, which was actually sustained. Obviously it is doubtful if the specimen could actually be able to develop a ductility of 9.0. Therefore, it was deemed more appropriate to use the values of $\mu_{\Delta,exp}$ as reported by the researchers for the load histories Type 2.

The plot in Fig. 4 shows that the most of the data points for $\mu_{\Delta,cal}$ for Type 1 load history are contained in a small band when they are plotted against ρ_A , while those for Type 2 load history are much more scattered. A second order curve fitting of Type 1 load history data points, yields

$$\mu_{\Delta,cal} = -12.86 \rho_A^2 + 15.54 \rho_A + 1 \quad (11)$$

with the goodness-of-fit index, R^2 , equal to 0.33. It is assumed here that the displacement ductility factor of 1.0 is attained for an un-confined section. The rather low R^2 is to be expected because the parameter ρ_A reflects several factors including the amount of transverse reinforcement, strength ratio, axial load level and tie configuration effectiveness, the latter being difficult to quantify precisely.

One may note that the plot of $\mu_{\Delta,exp}$ versus ρ_A (also shown in Fig. 4) agrees quite well with that of $\mu_{\Delta,cal}$ except for larger values of ρ_A , where there are few data points.

Clearly more data are definitely needed. As for Type 2 load history, no meaningful curve fitting is possible with the few sample points. Nevertheless, as expected, testing with displacement increments of $2\Delta_y$ seems to yield slightly higher ductility factor than testing with $1\Delta_y$ increments.

Energy Dissipation Capacity

The plots in Fig. 5 and Fig. 6 clearly indicate tendency for the normalized energy dissipation capacity and cumulative ductility ratio to increase nonlinearly with the effective confinement steel ratio. Results of second order curve fitting for the indexes based on the computed values of Δ_y (as explained earlier) are shown in the plots as solid lines. Assuming zero energy dissipation for a specimen without transverse reinforcement, which is a fairly good approximation, the equation of best fit, for displacement increments of $1\Delta_y$ is given by

$$E_{N,cal} = 83.67 \rho_A^2 + 49.58 \rho_A \quad (12)$$

and for the cumulative ductility ratio, the fitted curve is

$$N_{\Delta,cal} = 193.61 \rho_A^2 - 59.77 \rho_A + 20.33 \quad (13)$$

with corresponding R^2 values of 0.83 and 0.59, respectively.

For comparison, the fitted curves for the indexes based on the experimental values of Δ_y are also given as the dashed lines. Again, due to the paucity of data, only sample points for Type 2 load history are shown in the plots without any curve fitting performed. The fitted curves for normalized energy dissipation capacity based on the experimental and computed values of μ_Δ agree well, to within about 10% in the range of available data points. The same is true for the cumulative ductility ratio.

The limited data seem to suggest that the loading history with increments of $2\Delta_y$ is somewhat more severe than that with $1\Delta_y$ increments. For specimens with effective confinement steel ratio less than about 0.40, the energy dissipation capacity for Type 2

loading history may be smaller than that for Type 1 loading history by about 30% - 40%. The fact that the reduced value of μ_{Δ} results in an increase in the normalized energy dissipation capacity was earlier reported by Darwin and Nmai (1986). However, at larger values of ρ_A , e.g for highly confined columns, the effect of load history tends to be less significant. Due to the scatter in the test results and the paucity of data points, it is obvious that more tests are needed on highly confined specimens under small axial load levels.

As would be expected, there is a close correlation between the work index and cumulative ductility ratio as depicted in Fig. 7.

CONCLUSIONS

The following conclusions can be drawn:

1. For a given loading history, the displacement ductility factor, μ_{Δ} , and the energy dissipation parameters were found to correlate with the effective confinement reinforcement ratio. This is of practical significance in enabling prediction of displacement ductility and energy dissipation capacity for a given column, taking into account the effect of axial load and tie configuration effectiveness, among other basic variables.
2. The ductility factor for loading history Type 2 (with increments of $2\Delta_y$) tends to be somewhat higher than that for Type 1 (with $1\Delta_y$ increments) for the same effective confinement steel ratio. However, since the number of samples in the former case is small, further study is needed.
3. The load history with $2\Delta_y$ increments is slightly more severe than that with $1\Delta_y$ increments for moderate values of effective confinement steel ratio. The effect of load history tends to be less significant at higher values of the effective confinement steel ratio, pending further investigation.

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Table 1. Details of test specimens used in analyses

Reference (1)	Specimen (2)	A_g (m ²) (3)	Concrete Strength (Mpa) (4)	Longitudinal Steel		Lateral Steel			Axial Load Level $P/(f_c'A_g)$ (10)	Shear span-to-d epth ratio (11)	Tie Config. (12)	Load History Type (13)	k_e (14)
				f_y (Mpa) (5)	ρ_l (%) (6)	f_{yk} (Mpa) (7)	$A_{sh}/(s \cdot h_o)$ (8)	$A_{sh}/A_{sh,ACI}$ (9)					
Soesianawati et al. (1986)	UNIT 2	0.160	44.0	446	1.51	360	0.00601	0.55	0.30	4.00	RO	2(a)	0.58
	UNIT 3	0.160	44.0	446	1.51	364	0.00393	0.36	0.30	4.00	RO	2(a)	0.54
Watson et al. (1994)	UNIT 5	0.160	41.0	474	1.51	372	0.00579	0.58	0.50	4.00	RO	1(a)	0.57
	UNIT 6	0.160	40.0	474	1.51	388	0.00273	0.29	0.50	4.00	RO	1(a)	0.52
	UNIT 7	0.160	42.0	474	1.51	308	0.01111	0.91	0.70	4.00	RO	1(a)	0.53
	UNIT 8	0.160	39.0	474	1.51	372	0.00609	0.65	0.70	4.00	RO	1(a)	0.59
	UNIT 9	0.160	40.0	474	1.51	308	0.02051	1.76	0.70	4.00	RO	2(a)	0.71
Aziznamini et al. (1992)	NC-2	0.209	39.3	439	1.94	454	0.01152	1.01	0.21	3.00	RD1	1(a)	0.43
	NC-4	0.209	39.8	439	1.94	616	0.00650	0.68	0.31	3.00	RD1	1(a)	0.42
Wehbe et al. (1999)	A1	0.232	31.7	448	2.22	455	0.00323	0.52	0.08	3.83	RC	2(b)	0.46
	A2	0.232	27.2	448	2.22	455	0.00323	0.60	0.24	3.83	RC	1(a)	0.46
	B1	0.232	29.7	448	2.22	455	0.00428	0.73	0.09	3.83	RC	2(c)	0.53
	B2	0.232	28.1	448	2.22	455	0.00428	0.77	0.23	3.83	RC	1(b)	0.53
Sheikh and Khoury (1993)	AS-3	0.093	33.2	509	2.45	509	0.00844	1.44	0.60	4.83	RD2	1(a)	0.35
	AS-17	0.093	31.4	509	2.45	509	0.00844	1.52	0.77	4.83	RD2	1(a)	0.35
	AS-18	0.093	32.8	509	2.45	465	0.01501	2.36	0.77	4.83	RD2	1(a)	0.36
	AS-19	0.093	32.3	509	2.45	486	0.00691	1.15	0.47	0.00	RD2	1(a)	0.35

Table 2. Summary of displacement ductility and energy indexes

Reference (1)	Specimen (2)	Displacement ductility		Normalized dissipated energy		Cumulative ductility ratio		Work index	
		$\mu_{\Delta,exp}$ (3)	$\mu_{\Delta,cal}$ (4)	$E_{N,exp}$ (5)	$E_{N,cal}$ (6)	$N_{\Delta,exp}$ (7)	$N_{\Delta,cal}$ (8)	$I_{W,exp}$ (9)	$I_{W,cal}$ (10)
Soesianawati et al. (1986)	UNIT 2	6.0	9.0	20.1	30.3	19.1	28.8	17.4	26.2
	UNIT 3	5.8	8.6	11.8	17.4	12.7	18.7	11.9	17.6
Watson et al. (1994)	UNIT 5	5.7	5.4	30.7	29.3	30.2	28.9	27.7	26.5
	UNIT 6	4.0	4.5	17.3	19.5	18.9	21.3	17.2	19.4
	UNIT 7	3.2	3.2	13.4	13.1	10.4	10.2	9.7	9.5
	UNIT 8	3.9	3.5	11.3	10.1	10.3	9.2	9.6	8.6
	UNIT 9	9.1	7.3	61.5	49.5	41.0	33.0	38.3	30.8
Aziznamini et al. (1992)	NC-2	7.1	6.7	68.1	64.2	63.8	60.1	56.4	53.2
	NC-4	4.2	4.1	18.5	17.9	23.9	23.2	21.4	20.7
Wehbe et al. (1999)	A1	5.3	4.6	22.2	19.3	23.7	20.6	21.2	18.5
	A2	5.2	5.5	29.5	30.4	33.2	34.3	30.6	31.6
	B1	6.7	5.1	43.2	32.8	38.9	29.6	35.0	26.6
	B2	6.1	5.6	52.0	45.0	53.3	46.2	49.0	42.4
Sheikh and Khoury (1993)	AS-3	4.7	4.7	-	-	23.0	23.0	-	-
	AS-17	3.8	3.8	-	-	24.0	24.0	-	-
	AS-18	6.7	6.7	-	-	44.0	44.0	-	-
	AS-19	4.0	4.0	-	-	18.0	18.0	-	-

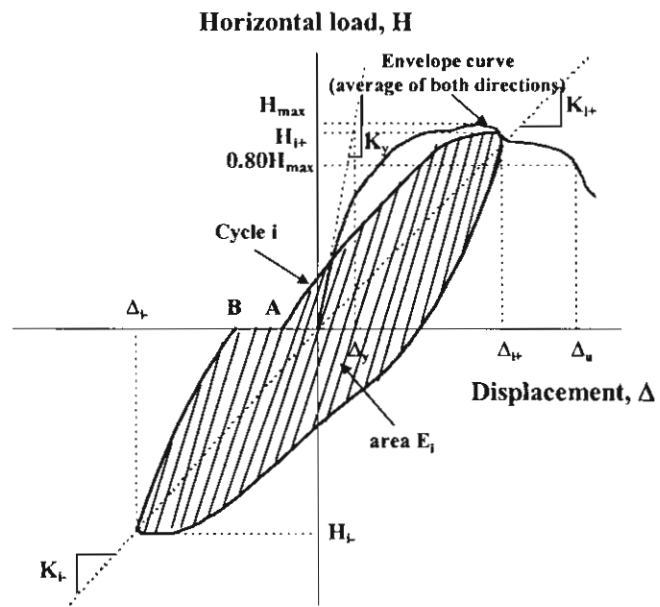


Fig. 1 Definition of yield displacement (Δ_y) and ultimate displacement (Δ_u)

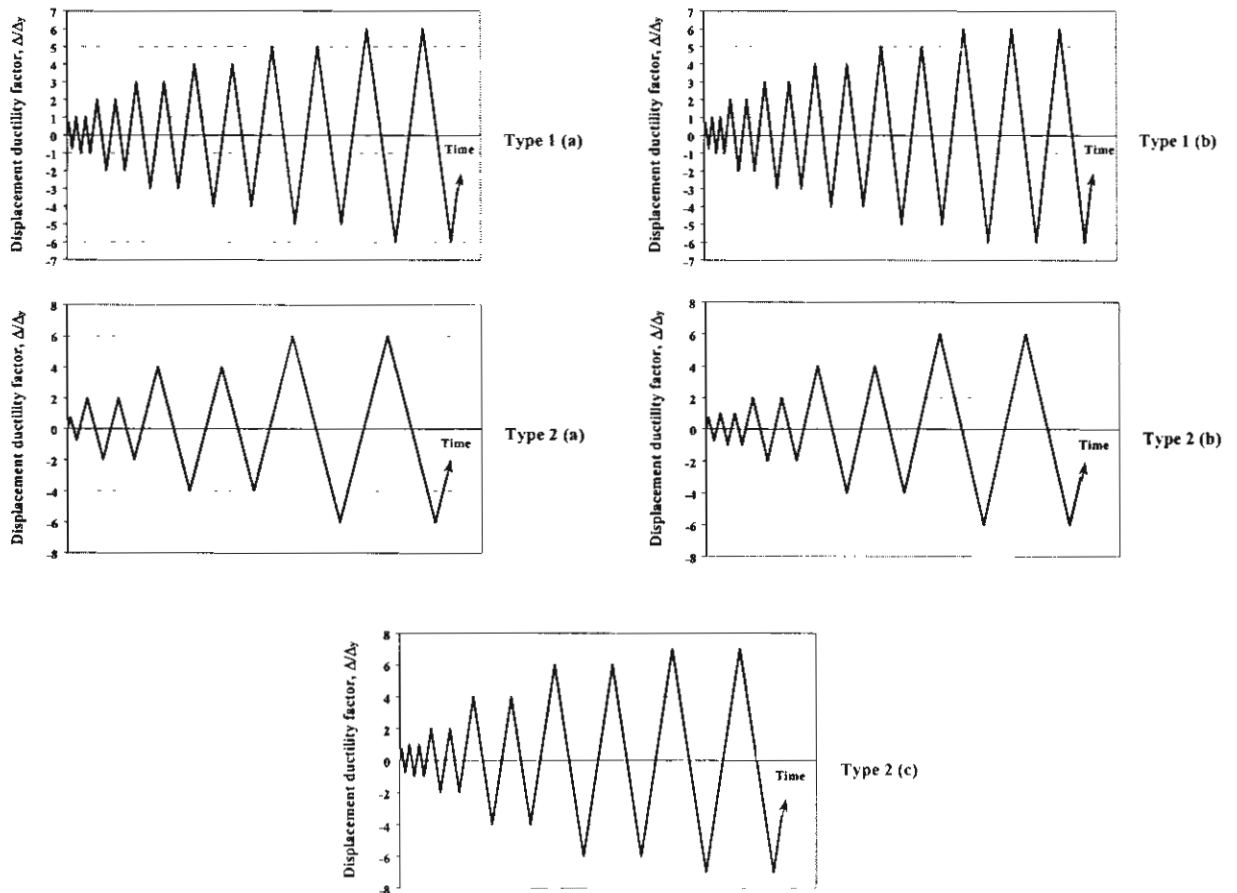


Fig. 2 Displacement histories

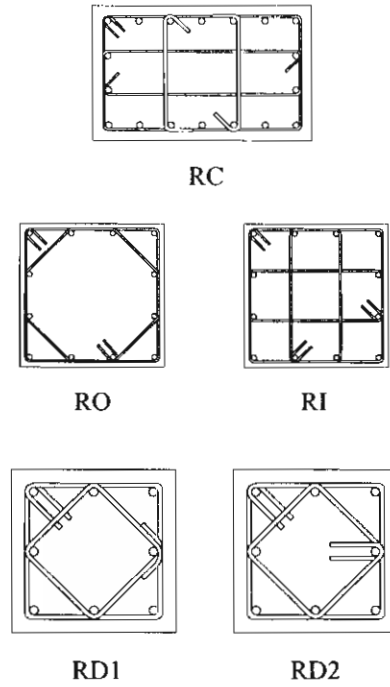


Fig. 3 Tie configurations

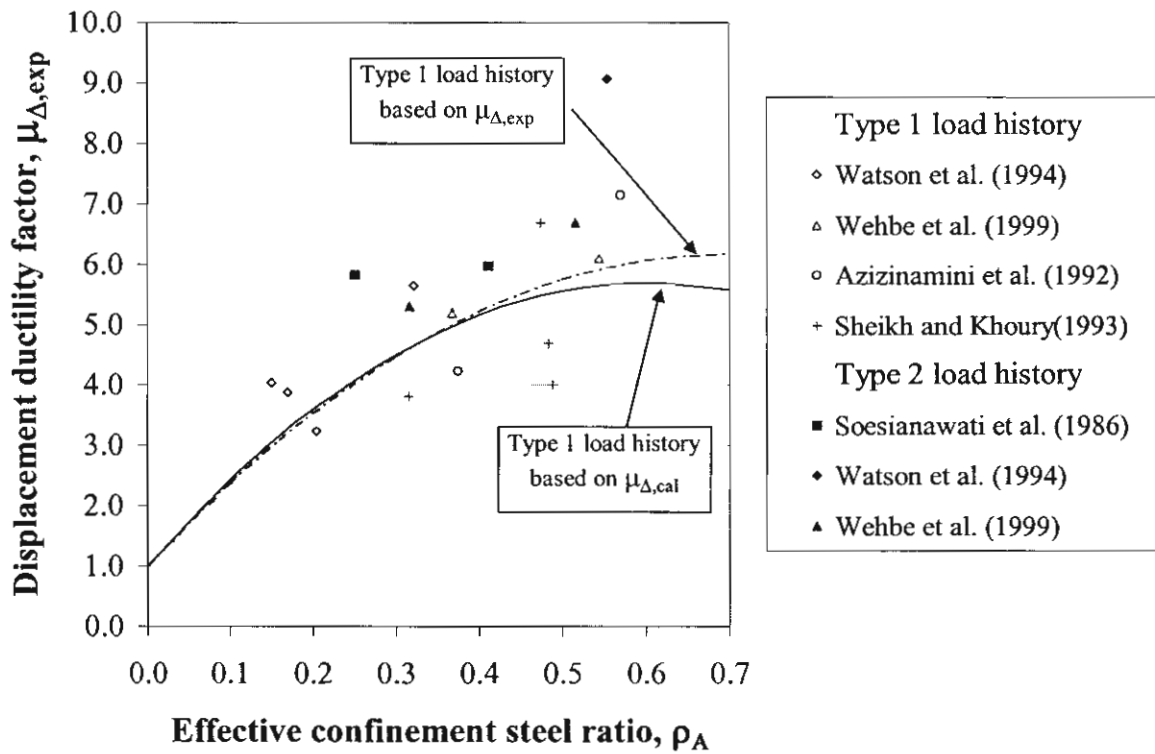


Fig. 4 Displacement ductility factor as function of ρ_A

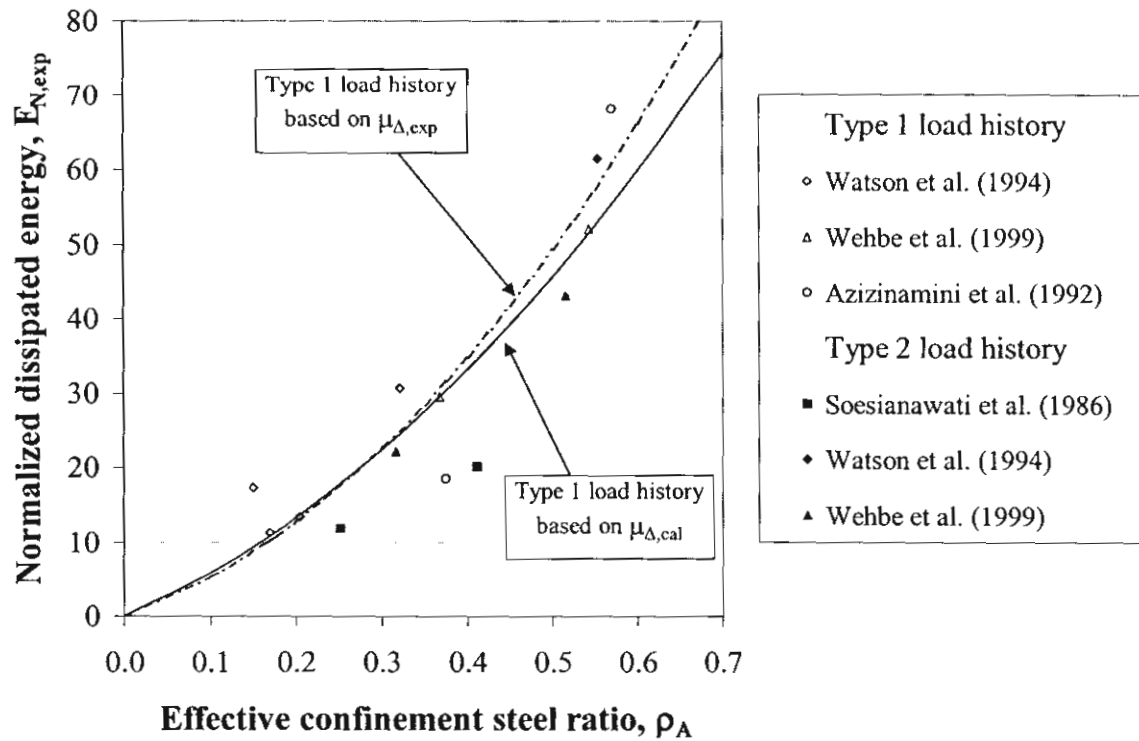


Fig. 5 Normalized dissipated energy as function of ρ_A

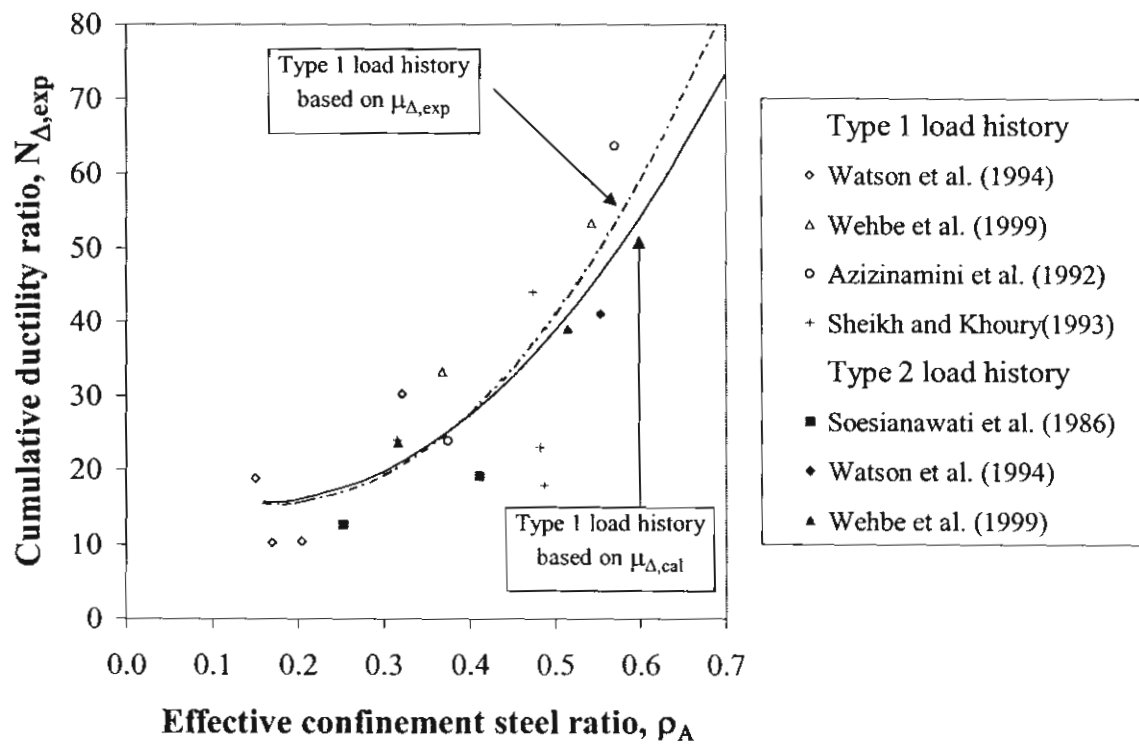


Fig. 6 Cumulative ductility ratio as function of ρ_A

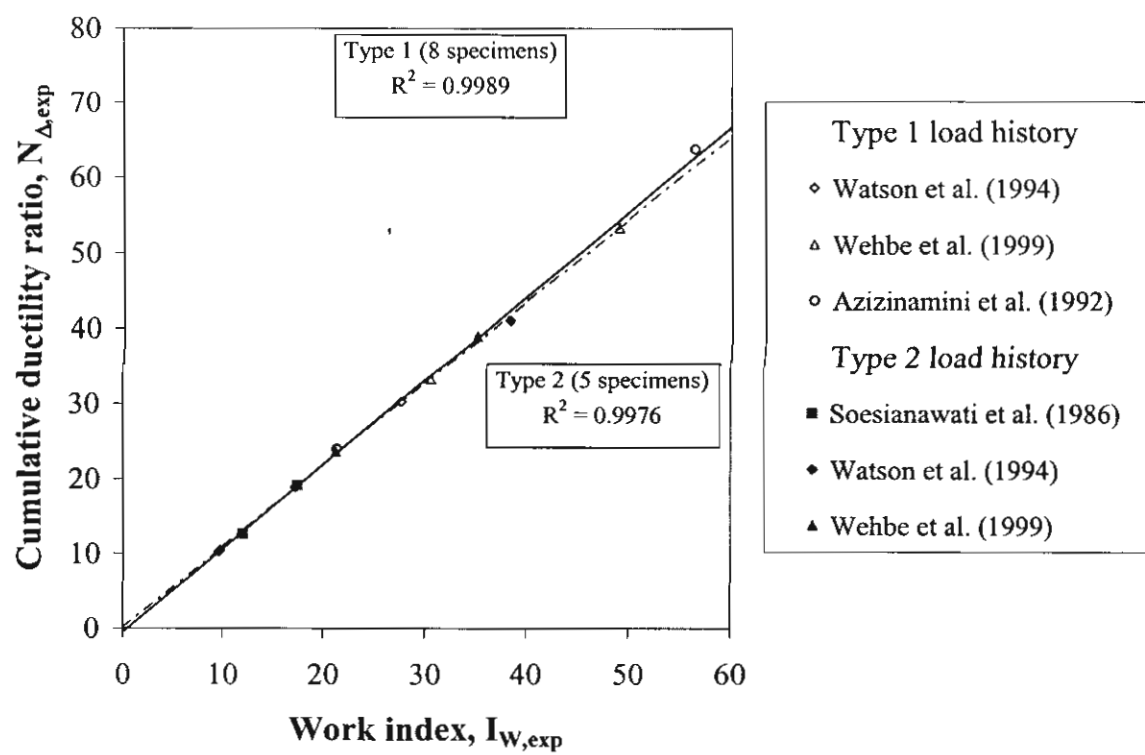


Fig. 7 Cumulative ductility ratio as function of I_W

RETROFIT OF STEEL MOMENT-RESISTING FRAMES SUBJECTED TO LONG-DISTANCE EARTHQUAKES CONSIDERING WELD FRACTURES

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It has been generally accepted that steel moment-resisting frames behave in a ductile manner under seismic excitations. However, during the 1994 Northridge earthquake, weld fractures at the beam-to-column connections occurred in many steel buildings. Such brittle failures obviously preclude the traditional ductile-behaviour assumption and have a significant effect on the responses of steel moment-resisting frames. In this paper, the performance of a friction damping system for retrofitting steel moment-resisting frames was investigated under long-distance earthquakes with frequency contents resembling the 1985 Mexico city (SCT), the 1995 Bangkok, or the 1977 Romania ground motions, all scaled to a peak ground acceleration of 0.17g. Responses of the building under the 1940 El Centro N-S component were also included for comparison. The results of the study show that a friction damping system can reduce the seismic responses significantly. The device can also prevent the total collapse and joint failures of the building equipped with friction dampers, while the one without dampers would collapse, even under a moderate peak ground acceleration of only 0.17g.

Keywords: weld fractures; joint failures; beam-column joints; long-distance earthquakes; friction dampers; energy ratio.

1. Introduction

As a result of the 1994 Northridge earthquake in California, many steel moment-resisting frames were seriously damaged. Brittle weld fractures in beam-to-column connections occurred in several steel structures. According to EERI [1997], two types of weld cracks can be distinguished. First is a fracture near the interface of the beam and column flange. This common fracture results in the severing of the beam flange from the column flange and passes through the weld or through the adjacent heat-affected material. Another type is a fracture through the

column. This type of fracture would be very dangerous if the crack extended all the way through the column which would reduce its compressive capacity. These fractures can violate the ductile-behaviour assumption of steel moment-resisting frames, which is usually presumed in an analysis, and they had an important effect on the seismic response of steel structures during the Northridge earthquake.

In recent decades, several analytical and experimental studies have led to the implementation of friction damping systems in many structures for retrofit, e.g., Pall and Marsh [1982], FitzGerald *et al.* [1989], Filiatrault and Cherry [1990], Grigorian *et al.* [1993], Pall A.S. and Pall R. [1996], Reinhorn and Li [1996]. The effectiveness of friction dampers in seismic retrofit has been widely recognized. However, most past studies did not consider the brittle weld fracture behaviour of steel moment-resisting frames, which can reduce the performance of steel structures. From the literature, it can be observed that there has been no research on retrofit of steel structures using friction dampers and taking into account the weld-fracture phenomena except the one by Filiatrault *et al.* [2001], which studied the performance of passive energy dissipating systems for retrofit of steel moment-resisting frames considering weld fracture effect. Those authors found that a friction damping system can significantly reduce responses of the building; however, major damage or total collapse in some cases still occurred and connection retrofit would still be required. The short duration, impulsive-type ground motions with average peak ground acceleration about 1 g, which represented the near field ground motions, were used. These findings, however, cannot be generalized for systems subjected to moderate ground motions induced by long-distance earthquakes, which may occur in many large cities in moderate seismic risk regions, such as Bangkok and Singapore.

This study investigates the performance of a passive friction damping system for retrofitting steel moment-resisting frames with weld fracture behaviour subjected to moderate ground motions from distant earthquakes. The effectiveness of friction dampers in reducing damage in structures is demonstrated through numerical examples obtained from nonlinear dynamic analyses. Only weld fracture at the beam-to-column interface was modeled into the studied building.

2. Building, Retrofit Scheme, and Model Assumptions

2.1. Building and retrofit scheme

The steel structure studied here is the same as considered by Filiatrault *et al.* [2001]. The building is a 6-storey steel moment-resisting frame, rectangular in plan, and is braced by two exterior moment-resisting frames. It is designed according to the 1994 UBC code requirements for a building located in seismic zone 4 on soil type S2. Gravity loads acting on the frame during the earthquake are assumed equal to 3.8 kPa from roof dead load, 4.5 kPa from floor dead load, 0.7 kPa from floor live load, and 1.7 kPa from weight of the exterior cladding. Steel grade A36 (nominal $F_y = 248$ MPa) is used for all members. Building details are shown in Fig. 1. For retrofit, chevron-brace members are introduced in the central bay of the two exterior moment-resisting frames as shown in Fig. 1. The steel section HSS 12 x 12 x 5/8 (300 mm x 300 mm x 15 mm) is used for all chevron-brace elements. Friction dampers are incorporated at one end of all bracing members.

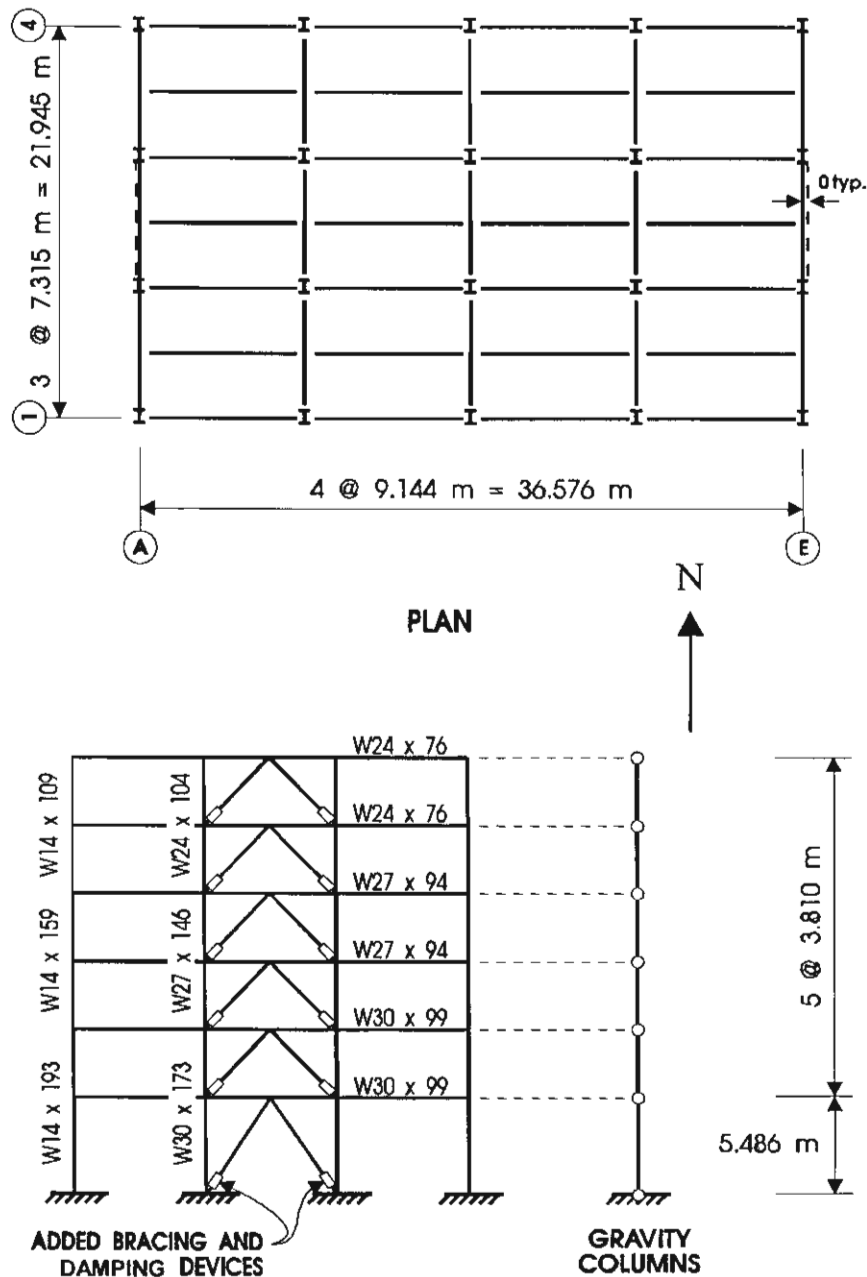


Fig. 1. Building and retrofit scheme

2.2. Model assumptions

Because of the symmetry of the studied building, only one exterior moment-resisting frame is modeled as a 2-D structure into the general-purpose nonlinear dynamic analysis program RUAUMOKO [Carr, 2000]. Floor slabs and architectural elements are excluded. The panel zones of the beam-column joints are assumed to have no panel shear deformation and yielding during strong excitations. This assumption represents the most critical condition for welded

beam-column joints because the hysteretic energy can only be dissipated through plastic hinging in the beams and the columns [Filiatrault *et al.*, 2001]. The inelastic response is assumed to concentrate in plastic hinges that could form at both ends of the frame members. Large displacement effect is also considered in the analyses. P- Δ effect from interior columns is included by introducing a pin-ended gravity column into the building model, which represents all interior columns. Total gravity loads acting on the interior columns are applied to the gravity column. Both the exterior frame and the gravity column are constrained to undergo the same lateral displacement at each floor, which represents a rigid floor diaphragm assumption. Bi-linear moment-curvature relation with a curvature-hardening ratio of 2% is assigned for all columns.

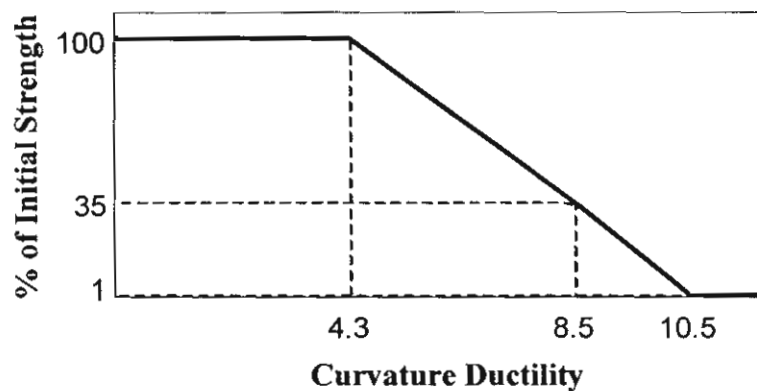


Fig. 2. Flexural strength degradation model

The flexural strength degradation model shown in Fig. 2 is introduced at both ends of all beams to account for the brittle behaviour of welded beam-to-column connections. Fracture initiates at curvature ductility of 4.3, corresponding to plastic rotation of about 0.01 rad in all beam sections of the studied structure, and the strength drops to 1% of the initial value at curvature ductility larger than 10.5. The strength degradation factor used here is the same as Filiatrault *et al.* [2001], which was obtained from a statistical review of different full-scale tests on beam-column joints, and it is assumed to be independent in positive and negative bending. Only fractures at the beam-to-column interfaces are considered. Then, an elasto-plastic moment-curvature relation is specified for all beam elements to prevent an increase in the flexural strength after fractures occur. Note that the connections are assumed to have no loss in shear capacity when weld fractures occur. An elasto-plastic force-displacement relation is also assigned for all friction-brace elements. The load at the yield plateau is specified by the slip load of each damper. The fundamental vibration period of the original building is 1.304 sec and is reduced to 0.649 sec after being retrofitted by the scheme described previously.

A mass proportional only damping model is adopted here because the rotational velocity at beam-to-column interface can increase significantly after fracture occurs. The large rotational velocity results in the increase in damping from the stiffness proportional portion of the global damping matrix, and this can violate the equilibrium condition at beam-column joints after weld fracture occurs [Filiatrault *et al.*, 2001]. The mass-proportional damping coefficient is taken as 0.5 sec^{-1} . The time step for nonlinear dynamic analyses is 0.0005 sec for all cases and a tolerance of 1% in the residual force vector is imposed for Newton-Raphson iteration to track the sudden

changes in stiffness due to joint fractures. Note that the slip loads of the friction dampers in the 2nd to 6th floors are 80% of slip load in the 1st floor. It is also assumed that the devices have sufficient slip lengths, so that no impact can occur.

The concept of Coulomb friction was adopted here to describe the hysteretic behaviour of the friction dampers, which can be formulated as:

$$F_S = \mu N \quad (1)$$

where F_S is a slip load of the damper, N is a normal force acting on the friction surface, and μ is a coefficient of friction. The friction devices will slip at a predetermined load exhibiting the well-known rigid-plastic hysteretic behaviour.

3. Earthquake Ground Motions

Accelerograms and pseudo-acceleration response spectra at 5% damping of all studied excitations are shown in Fig. 3. Three different long-distance earthquakes are considered here. First is a well-known long-distance earthquake, the 1985 Mexico city (SCT, E-W component) record. The SCT excitation was recorded on September 19, 1985 in Mexico city. The event's epicentral distance was about 400 km. It had a significantly narrow frequency band with a predominant period of 2.0 sec and peak ground acceleration about 0.17 times the gravitational acceleration (g). It is also a long-duration strong ground shaking which can have a significant effect on structures. To reduce the calculation time, only the 60-second range of the SCT record containing the peak acceleration value will be used in analysis. Second is the scaled 1995 Bangkok excitation. It was recorded on July 12, 1995 at the basement level of the Baiyoke tower whose fundamental vibration period is 2.5 sec. This event resulted from an earthquake whose epicenter was about 900 km away from Bangkok. Its predominant period is about 2.3 sec and it also contains other significantly longer and shorter period components. Its peak ground acceleration was scaled to the same value of SCT record.

The last long-distance ground motion is the scaled 1977 Romania (Bucharest, N-S component) earthquake. It was recorded on March 4, 1977 in the basement of the Research Institute building in Bucharest, Romania. The epicentral distance was about 150 km. The event had a narrow frequency band with a predominant period about 1.2 sec. Its peak ground acceleration was also scaled to 0.17 g . The studied building was also subjected to the 1940 El Centro N-S component excitation which had been used extensively in many past studies. This signal was considered here for comparison purposes. Note also that the peak ground acceleration of the El Centro earthquake is about 2 times that of the other three records.

4. Numerical Results

In this section, the responses of the building under all four excitations are compared. The response parameters of interest are the peak interstory drifts, the number of yielded members and joint fractures, and the energy input and dissipated. The variation of these parameters with the slip load of the devices is investigated. The slip load of each device is varied between zero and

buckling strength of bracing member. Responses of the original building excluding weld fracture behaviour are also shown for comparison.

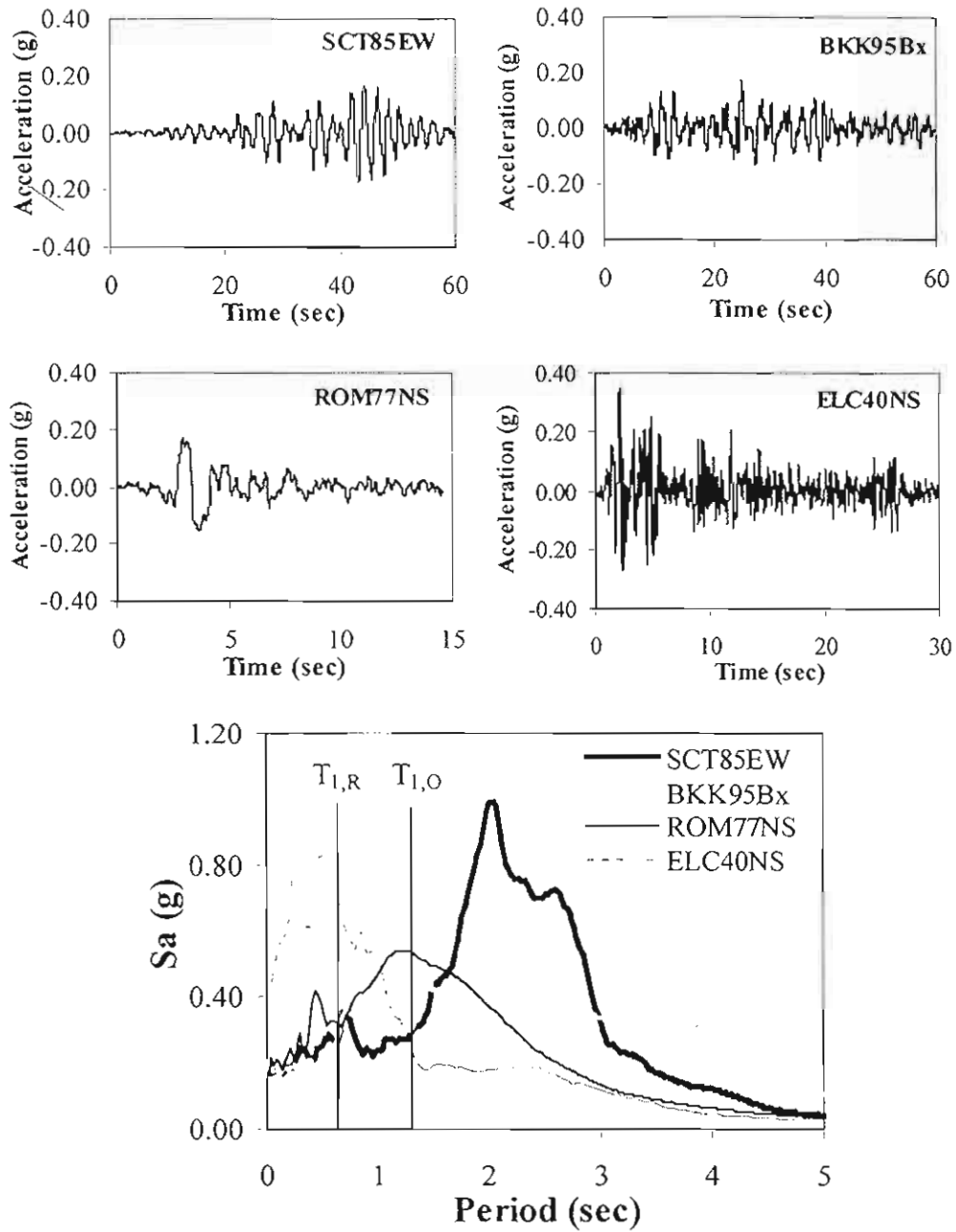


Fig. 3. Accelerograms and pseudo-acceleration response spectra at 5% critical damping

4.1. Peak interstory drift

The interstory drift can be an indicator for both structural and nonstructural damage. According to the performance criteria defined by the NEHRP seismic rehabilitation guidelines [ATC, 2000] for steel moment-resisting frames, maximum drift levels are 0.7% for immediate occupancy, 2.5% for life safety, and 5% for collapse prevention. Therefore, in this study, the 5% interstory drift is used as the collapse criterion. Fig. 4 shows the variation of peak interstory drift in each floor of the original structure neglecting weld fracture effect. From the figure, the original building can survive all four earthquakes considered. The maximum interstory drifts, which are found in the first floor are 1.51%, 1.46%, and 0.81% resulting from the SCT, Romania, and El Centro excitations, respectively, while the maximum interstory drift of 1.14% is found in the second floor when the building is subjected to the Bangkok signal.

Fig. 5 shows the variation of peak interstory drift in each floor with the normalized slip load of the structure including weld fracture effect. The slip load is normalized by the buckling strength of the ground floor bracing element which equals to 3811 kN. The non-retrofitted building including joint fracture collapses under all excitations which is indicated by a peak storey drift of more than the 5% drift limit. Interestingly, even moderate long-distance earthquakes which have a peak ground acceleration only about half of the El Centro record can lead to the total collapse of the structure. This result indicates that weld fractures have a large effect on the performance of the structure and they must be taken into account.

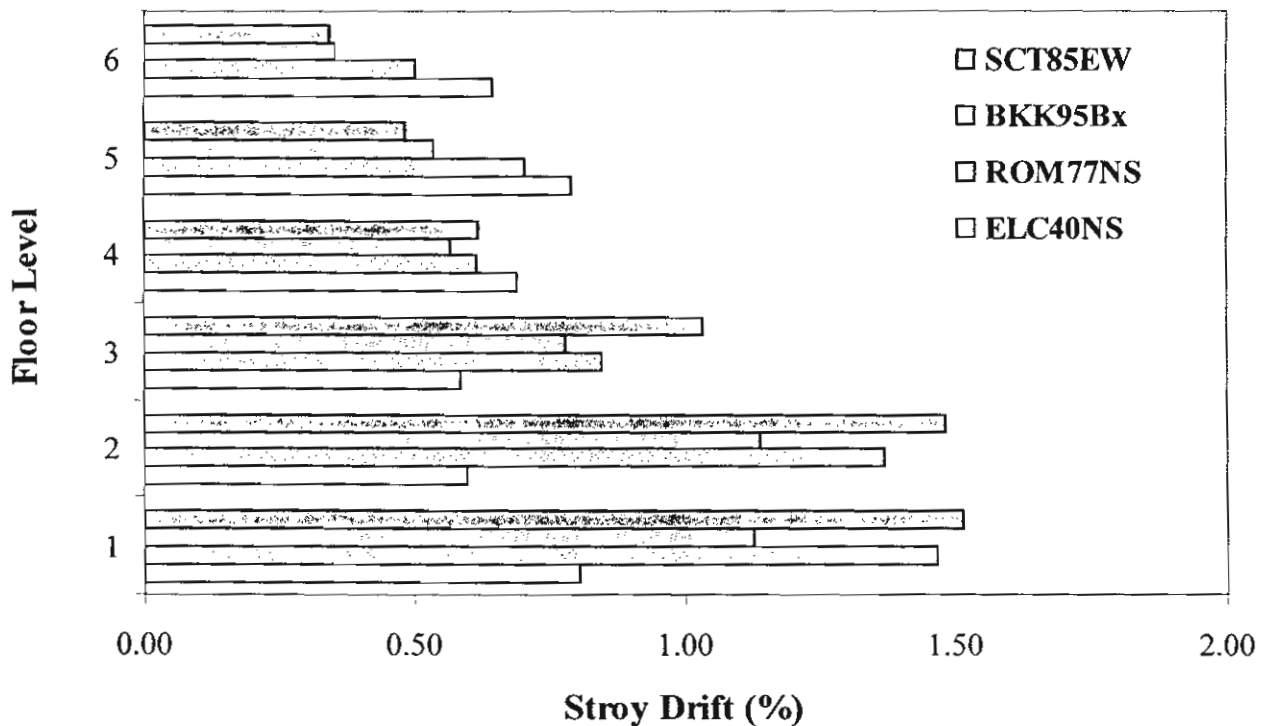


Fig. 4. Variation of peak interstory drift of the non-retrofitted building neglecting weld fracture effect

For a retrofitted structure including the effect of weld fracture, the building survives the El Centro earthquake at any slip load values, even under a very low normalized slip load of only 2.3%. However, it collapses under the SCT, Bangkok, and Romania ground motions as joint fractures take place at normalized slip loads less than 20.7%, 25.6%, and 19.7% for SCT, Bangkok, and Romania records, respectively. It is important to note that at a slip load close to these critical values, damages increase very rapidly. It should also be noted that larger earthquakes would require a higher critical slip load for collapse prevention. This is of practical significance in selecting the slip load of a friction damper to be sufficiently larger than the critical threshold values, and at the same time not too close to the buckling strength or the yield strength of the connecting elements; otherwise, the friction devices may not be effective to protect the structure. Past the threshold slip loads, responses of the buildings are sharply reduced. Peak interstory drifts are kept below 1% in all cases, which indicate that the retrofitted structure can survive all three long-distance excitations considered here. Note that there is an optimum slip load value in the case of the building subjected to the El Centro excitation. The optimum normalized slip load is 19.7% which is similar to the results obtained by Filiatrault *et al.* [2001]. On the other hand, for the three long-distance earthquakes, it seems that the performance of building improves for increasing slip loads. This would be caused by the characteristics of these studied accelerograms that have a frequency content concentrating in the low frequency zone. When joint fractures take place, the building will lose its stiffness and the resonant effect can occur. Then the additional stiffness induced by bracing system will play a more important role. Therefore, at a large slip load value, the devices are more difficult to slip. Then the building will be stiff and will respond for a longer time in the low spectral value zone resulting in a better performance.

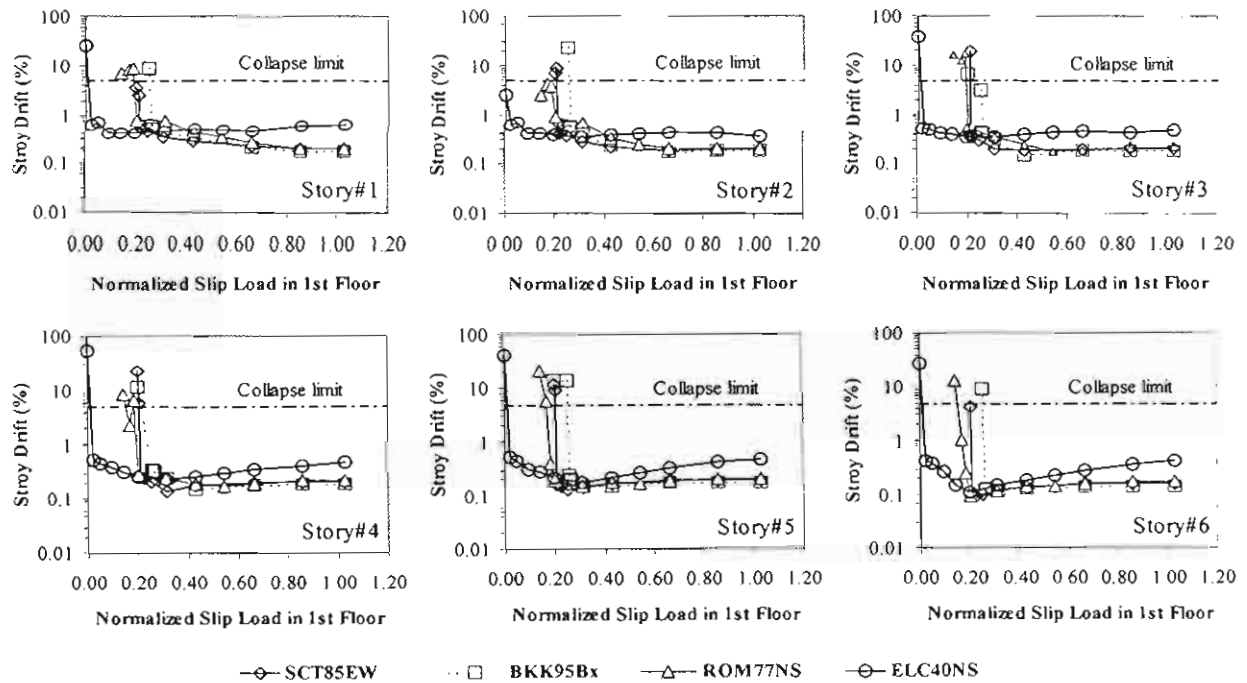


Fig. 5. Variation of peak interstory drift with normalized slip load of the building including weld fracture effect

However, if the slip loads reach the buckling or yield strength of the bracing elements, the performance of the system will decrease due to the buckling phenomena, or the failure of the bracing members. Note that buckling of the bracing members at the normalized slip load of 100% is neglected in this study.

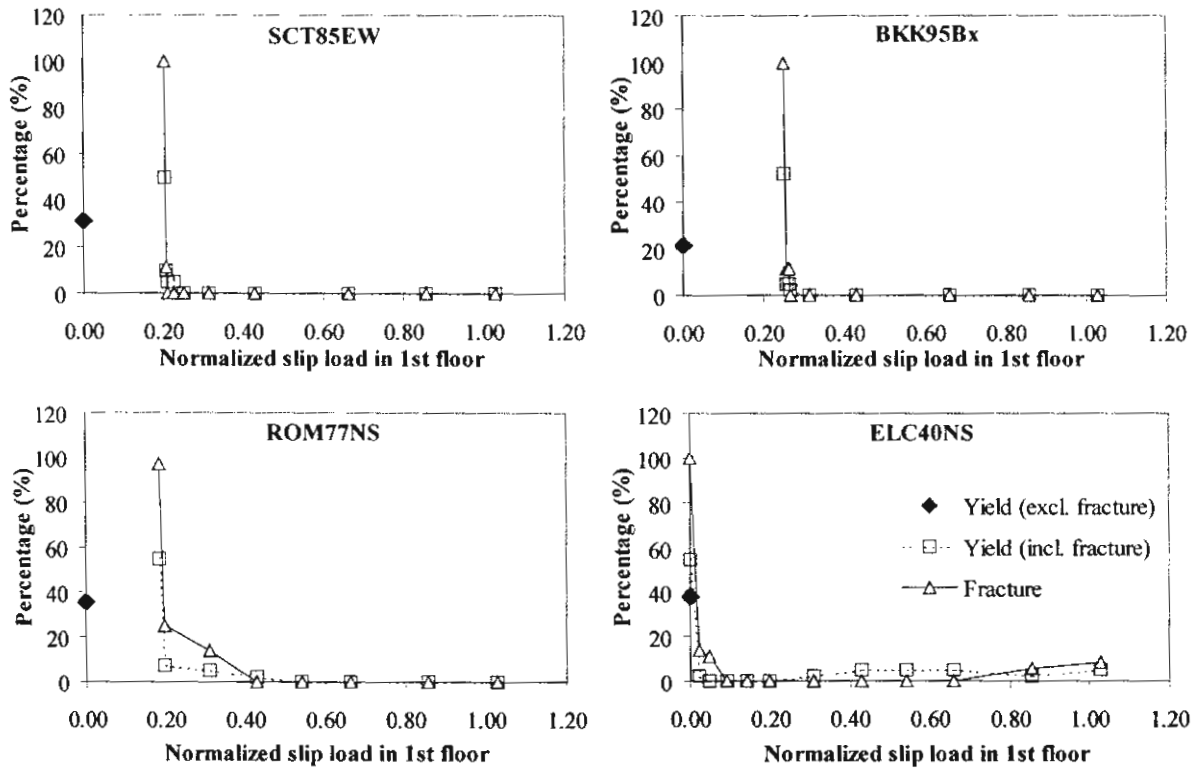


Fig. 6. Variation of the percentage of the yielded members and joint fractures with normalized slip load

4.2. Damage to the structure

Fig. 6 shows the percentage of yielded members and joint fractures for the original building neglecting weld fracture and for the retrofitted structure including weld fracture at each slip load value. For the numbers of yielded members, all members except the bracing elements and the gravity column are counted corresponding to a total of 42 members. The numbers of joint fractures are presented for each beam's end resulting in a total of 36 joints. For the original structure excluding weld fracture, the percentages of yielded members are 31%, 21.4%, 35.7%, and 38.1% when subjected to SCT, Bangkok, Romania, and El Centro ground motions, respectively. Although the building can survive all considered earthquakes, damages occur in many structural members which can lead to a high repair cost. Retrofit would be required for this structure.

For the retrofitted structure considering weld fracture characteristics, past the critical slip load value, the friction devices can reduce damages significantly. At the normalized slip load larger than 25%, 31.2%, and 54.2% when subjected to SCT, Bangkok, and Romania accelerograms,

respectively, all structural elements remain elastic and no joint failures occur. Similarly, for the retrofitted structure subjected to the El Centro record, the building remains elastic at the normalized slip load between 9.2% and 19.7%. Under all long-distance excitations, the studied building exhibits a sudden increase in damage at slip load less than the certain critical values indicated by failures of all joints in the building. Selection of the device's slip load would be very important for the structure having a possibility of weld fracture.

4.3. Energy ratio

In this section, the amount of energy dissipated by means of friction dampers for all analyses are presented. The results are shown in terms of the energy ratios, *INR* and *FDR*, defined as follows:

$$INR = \frac{E_I}{V_{BY} \Delta_Y} \quad (2)$$

$$FDR = \frac{E_{FD}}{V_{BY} \Delta_Y} \quad (3)$$

where *INR* is the seismic input energy ratio; *FDR* is the friction-damper dissipated energy ratio; E_I is the seismic input energy; E_{FD} is the energy dissipated by all friction dampers; V_{BY} and Δ_Y are the yield base shear and the top-floor yield displacement of the braced structure without dampers, respectively. Derivations of E_I and E_{FD} are given in Appendix A.

Fig. 7 shows a relation between base shear and top-floor displacement obtained from pushover analysis of the braced building, without dampers, including weld fractures. The same bracing system as in the previous numerical studies is employed. Lateral force distribution according to the UBC 1997 code is adopted here. The values of lateral forces in each storey as fraction of base shear are shown in Table 1. A monotonic increasing load is applied to the structure with linear increment of 200 kN/sec until the ultimate load is reached. The same time step as used in the previous numerical studies is chosen. From the analysis, the yield base shear is 6448 kN and the yield top-floor displacement of the brace structure is 79.0 mm.

Table 1. Lateral force distribution factors as a fraction of base shear for braced structure

Storey level	Height (m.)	Weight (kN)	Distribution factor
6	24.536	1815.53	0.2123
5	20.726	2514.80	0.2484
4	16.916	2514.80	0.2028
3	13.106	2514.80	0.1571
2	9.296	2514.80	0.1114
1	5.486	2599.13	0.0680

Note: Fundamental period of braced structure = 0.649 sec

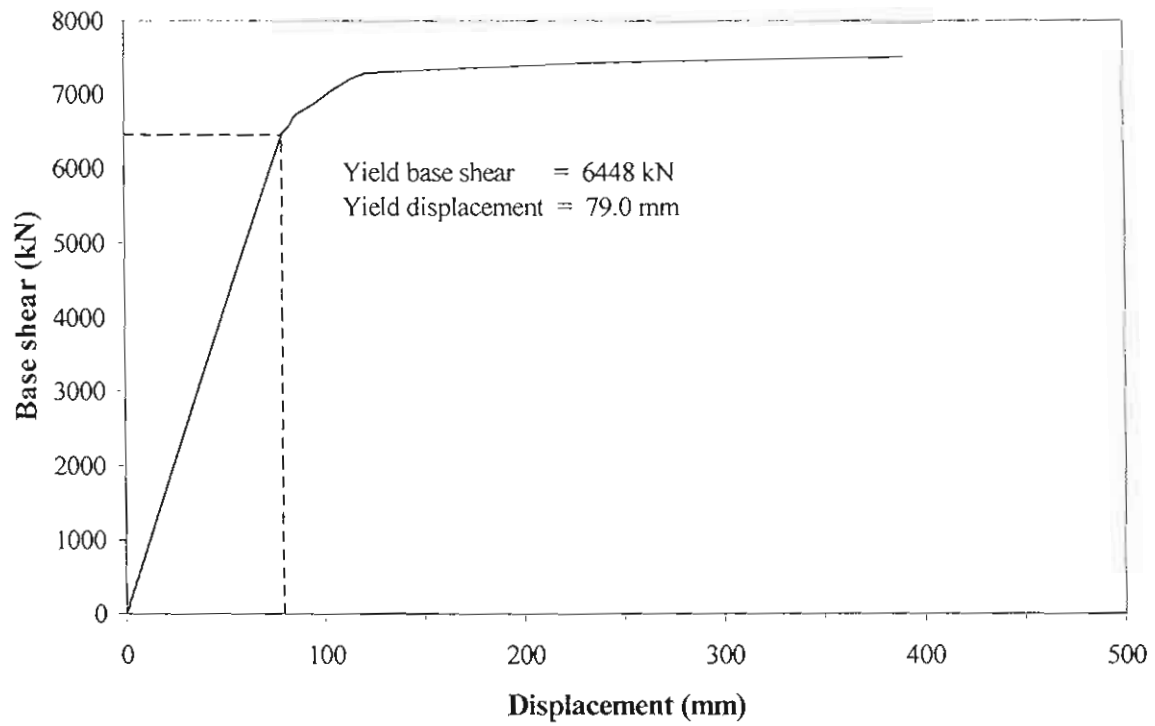


Fig. 7. Pushover analysis of the braced building without dampers including weld fracture effect

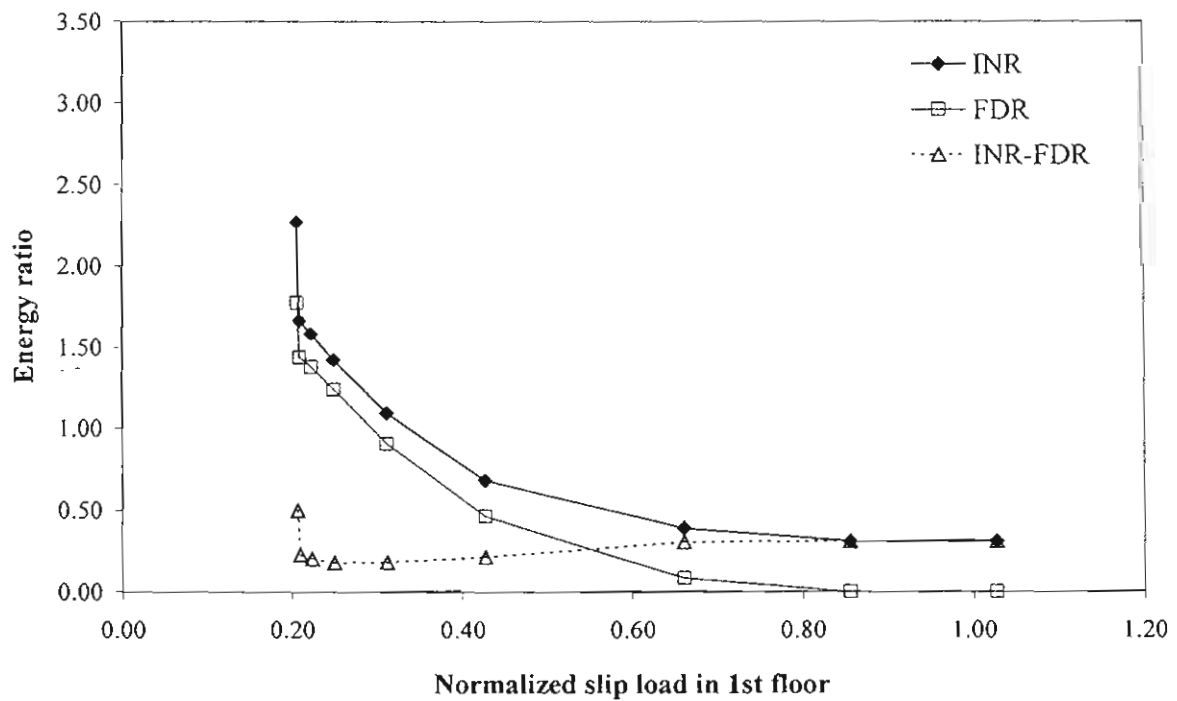


Fig. 8. Variation of energy ratio with normalized slip load : SCT85EW

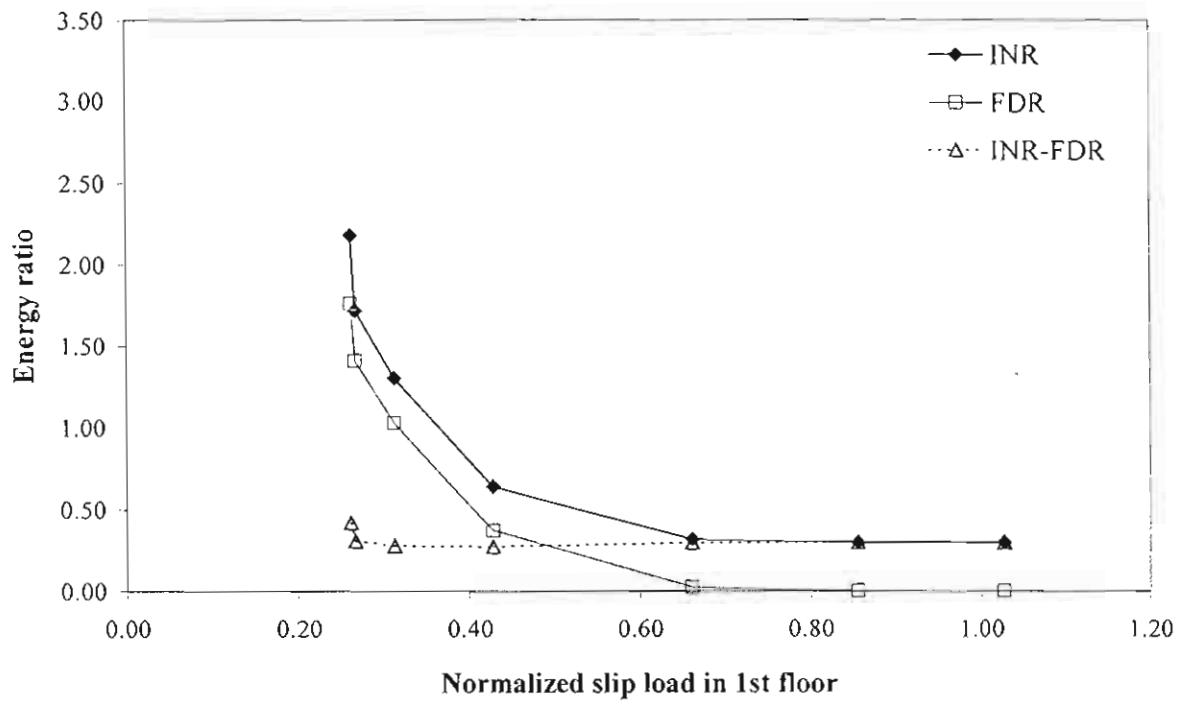


Fig. 9. Variation of energy ratio with normalized slip load : BKK95Bx

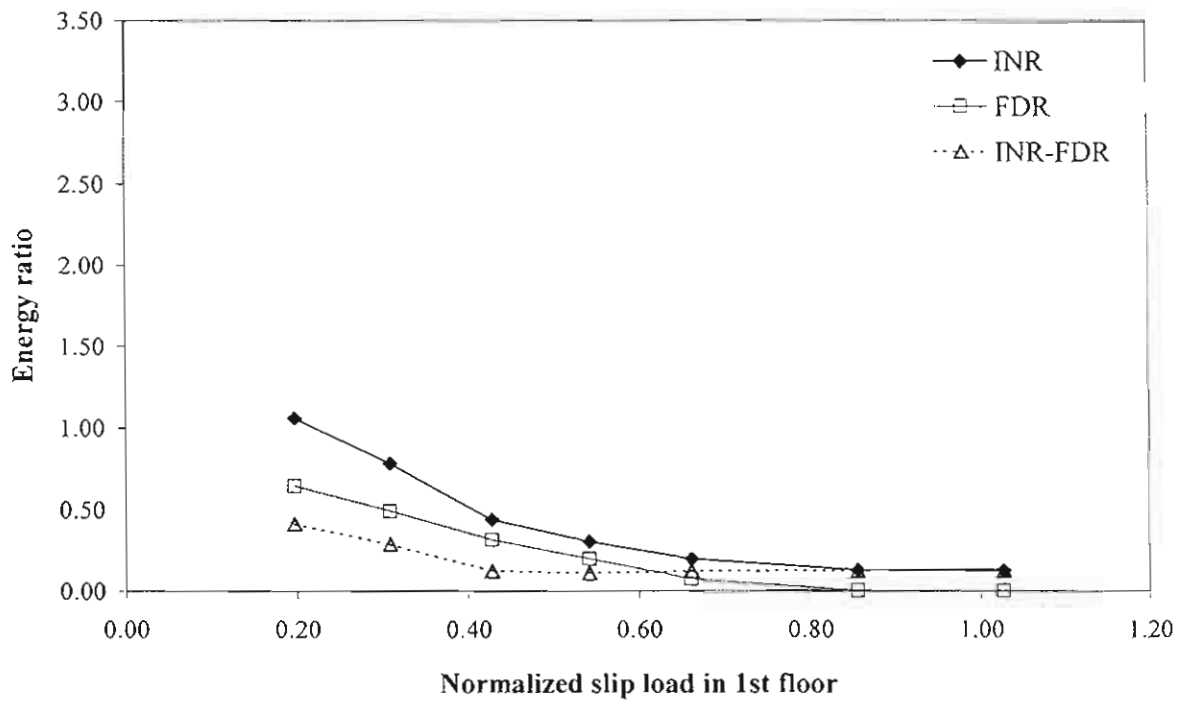


Fig. 10. Variation of energy ratio with normalized slip load : ROM77NS

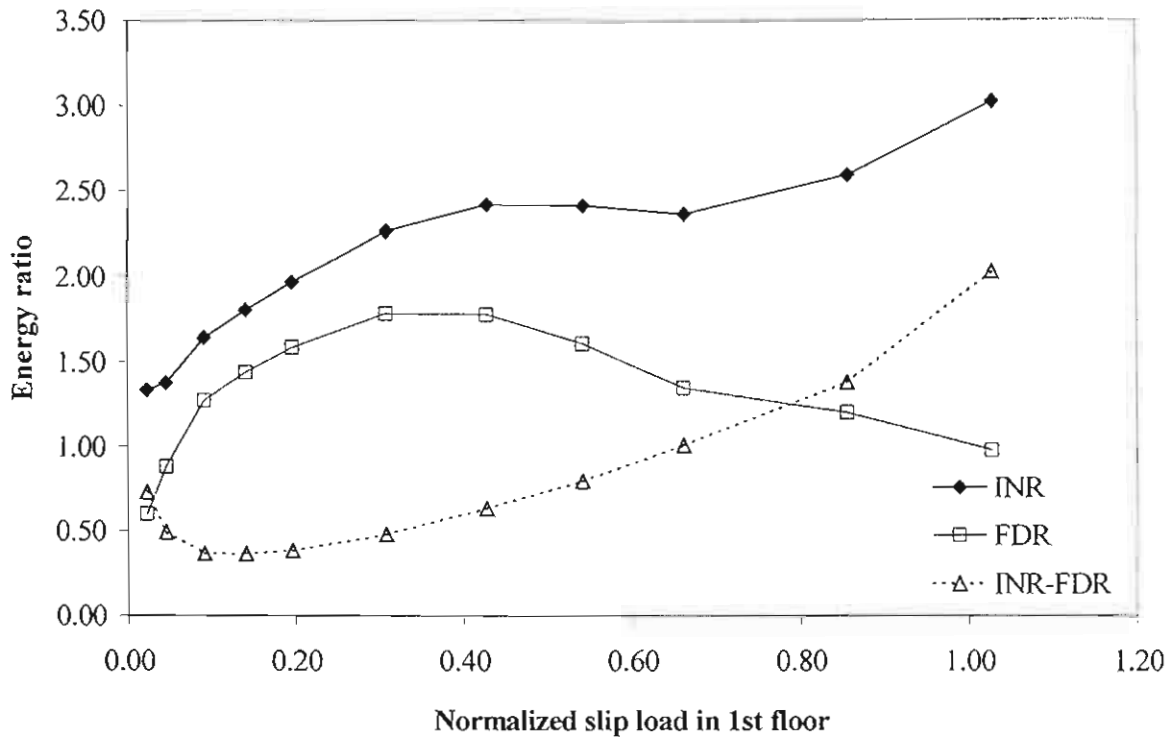


Fig. 11. Variation of energy ratio with normalized slip load : ELC40NS

Figs. 8 to 11 show the variations of the energy ratios with the normalized slip load in the first floor for building subjected to each excitation. The difference between the seismic input energy ratio (*INR*) and the friction-damper dissipated energy ratio (*FDR*) indicates the amount of energy dissipated by inherent damping and damage of the structure itself. It can also be considered as the performance index for the added friction dampers. Again, past the critical slip load, friction dampers can significantly reduce the energy fed to the primary structure. At the optimum points, the devices can absorb 87%, 58%, 65%, and 80% of the seismic input energy resulting from SCT, Bangkok, Romania, and El Centro signals, respectively. For El Centro, as expected, the performance of the damper decreases with increasing slip loads. For long-distance earthquakes at large values of slip load, the energy dissipated by the dampers vanishes indicating that the added dampers are not utilized. This is compensated by reduction of the seismic input energy because at high values of slip load, the system behaves as a fully braced structure resulting in the shift of a vibration period to the short period end of the response spectra which has low spectral values as shown in Fig. 3.

From the results obtained, it seems that only the added stiffness from the braces is sufficient for the building to resist the long-distance earthquakes considered. However, it should be noted that the buckling phenomena of bracing members are neglected in this study. If both the brace buckling and joint fractures occur, the stiffness of the system will sharply reduce and this could lead to a total collapse of the whole structure. Hence, addition of friction dampers will have the advantage of preventing buckling or damage to the bracing elements and give nearly equal or better performance compared to the traditional bracing system.

5. Conclusions

The performance of a friction damping system for retrofitting steel moment-resisting frames with weld fracture behaviour was investigated under three long-distance earthquakes: the 1985 Mexico city (SCT), the scaled 1995 Bangkok, and the scaled 1977 Romania ground motions. Responses of the building under the 1940 El Centro record were also shown for comparison purposes. The following conclusions can be drawn from this study:

1. The original building with joint fracture effect neglected can survive all considered excitations; however, retrofit would still be required.
2. The effect of weld fracture can lead to the collapse of the non-retrofitted structure under all excitations considered, even under moderate long-distance earthquakes which have a peak ground acceleration only about half of the El Centro accelerogram.
3. The building including joint fracture effect can survive all considered earthquakes with the addition of friction dampers at a certain critical slip load value.
4. It is important to pay attention to selecting the damper slip load to be sufficiently far away from the critical value and at the same time not too close to the buckling or yield strength of the connecting members.
5. Addition of friction dampers will have the advantage of preventing buckling or damage to the bracing elements and give nearly equal or better performance compared to the traditional bracing system.

Acknowledgements

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Appendix A : Energy Formulation

Consider a multi-degree-of-freedom system equipped with friction dampers. Denoting $\mathbf{u}$ as the vector of displacements relative to the base and $u_g(t)$ is the horizontal ground displacement at any time t , the equations of motion can be written as:

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{F}_s(t) + \mathbf{F}_{FD}(t) = -\mathbf{M}\mathbf{e}_g\ddot{u}_g(t) \quad (\text{A.1})$$

where $\mathbf{M}$ and $\mathbf{C}$ are the mass and damping matrices of the structure; $\mathbf{F}_s$ is the vector of the resisting forces provided by the main structure and $\mathbf{F}_{FD}$ is the vector of forces provided by the friction dampers. $\mathbf{e}_g$ is the displacement influence coefficient vector associated with a unit support displacement defined as follows:

$$\mathbf{u}_t(t) = \mathbf{u}(t) + \mathbf{e}_g u_g(t) \quad (\text{A.2})$$

where $\mathbf{u}_t(t)$ is the vector of total displacements.

Integrating the equations of motion with respect to the relative displacement vector $\mathbf{du}$ over the entire range of displacements, the energy equation can be obtained as follows:

$$\int \mathbf{du}^T \mathbf{M} \ddot{\mathbf{u}} + \int \mathbf{du}^T \mathbf{C} \dot{\mathbf{u}} + \int \mathbf{du}^T \mathbf{F}_s + \int \mathbf{du}^T \mathbf{F}_{FD} = - \int \mathbf{du}^T \mathbf{M} \mathbf{e}_g \ddot{u}_g$$

$$E_K + E_D + E_S + E_{FD} = E_I \quad (\text{A.3})$$

The right side of Eq. (A.3), E_I , is the relative seismic energy input to the structure. For the terms on the left side, E_K is the kinetic energy of the mass associated with its motion relative to the ground; E_D is the energy dissipated by the inherent damping of the main structure; E_S is the total strain energy caused by deformations of the main structural elements and E_{FD} is the total energy dissipated by the added friction dampers. Based on these energy quantities, Eq. (A.3) is a statement of energy balance for the system.

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Vibration control of structures under harmonic excitation using semi-active mass damper

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Abstract:

A semi-active mass damper (SAMD) in which its damping can be varied is proposed to control the vibration of the structures under harmonic excitation. The objective is to produce a control system that provides a significant improvement over the conventional passive mass damper (TMD), but without the substantial penalties in cost, weight, and complexity as active damper. An optimal control law is formulated. A situation in which the damper can not vary beyond the certain range is also taken into account. The peak and steady-state amplitude responses are investigated in frequency domain. The obtained results are compared with those of TMD and it is found that the proposed SAMD can significantly reduce the peak response and also effectively suppress the steady-state response for broader band of excitation frequencies. The latter improvement is equivalent to the addition of mass about 300% on TMD.

Introduction

Structural vibration control has been one of the primary issues for some years. Active control is very efficient, but its implementation is very limited due to various mechanical and economical constraints. From this point of view, semi-active control is a promising alternative since it requires much less energy and it can potentially more efficient than passive control.

Karnopp, et. al. (1974) proposed the semi-active damping concept by using a variable damping device to generate the control force for vibration isolation. This semi-active control has been extensively studied in the field of automotive suspension control. To date, it has been successfully applied to a board class of vehicle vibration isolation problems ranging from military tanks to high-speed cars. An extension of the semi-active damping concept to structural control is found in the study of Hrovat, et. al. (1983). A TMD with an augmented variable damping device was proposed as a semi-active TMD for vibration control. Based on the numerical results of vibration control of a building subjected to a specific wind loading, the similar steady-state performance of semi-active TMD and active TMD was reported. However, there was no explicit study on the control performance of semi-active mass damper under harmonic excitation which is the common case in structural or mechanical control.

Optimal Semi-active Mass Damper

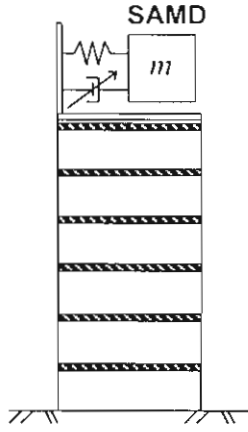


Figure 1. Structure-SAMD system

General Solution

A model representing a structure attached by the semi-active mass damper is shown in figure 1. This model can be expressed as a bilinear system in which the equations of motion are written in the following state-space description as

$$\dot{\mathbf{x}}(t) = \mathbf{A} \mathbf{x}(t) + \mathbf{B} (\mathbf{x}^T(t) \mathbf{T}) \mathbf{v}(t) = \mathbf{A} \mathbf{x}(t) + \mathbf{B} \mathbf{u}_{semi}(t) \quad (1)$$

where $\mathbf{x}(t)$ is a state vector, $\mathbf{A}$ is a characteristic matrix, $\mathbf{B}$ is a control contribution matrix, $\mathbf{T}$ is a transformation matrix, $\mathbf{v}(t)$ is an adjustable parameter vector, and $\mathbf{u}_{semi}(t) = \mathbf{x}^T(t) \mathbf{T} \mathbf{v}(t)$ is a semi-active control force produced by the variation of $\mathbf{v}(t)$. The objective is to find the parametric control $\mathbf{v}(t)$ that minimizes the well-known quadratic performance index

$$J_{semi} = \int_0^{\infty} [\mathbf{x}^T(t) \mathbf{Q} \mathbf{x}(t) + \mathbf{u}_{semi}^T(t) \mathbf{R} \mathbf{u}_{semi}(t)] dt \quad (2)$$

subjected to the dynamic constraint, Eq.(1).

As obtained in the determination of optimal semi-active vehicle suspension system (Tseng and Hedrick 1994), it is convenient to first consider the optimal solution for fully-active system case. Then the corresponding semi-active optimal solution is determined by minimizing the variation of performance index which is deviated from its reference optimum due to the introduction of semi-active system. It is, therefore, helpful to relate the performance index of general semi-active system to that of the reference active system as

$$J_{semi} = J_{active} + \int_0^{\infty} [(\mathbf{u}_{semi}(t) - \mathbf{u}_{opt}(t))^T \mathbf{R} (\mathbf{u}_{semi}(t) - \mathbf{u}_{opt}(t))] dt \quad (3)$$

where J_{active} is a constant function of initial state and $\mathbf{u}_{opt}$ is the optimal control force of the reference active system. Instead of directly minimizing the whole integration, the control law is obtained by minimizing the instantaneous error resulted from the difference $(\mathbf{u}_{semi}(t) - \mathbf{u}_{opt}(t))$. This leads to the minimization of the function

$$L(t) = (\mathbf{u}_{semi}(t) - \mathbf{u}_{opt}(t))^2 \quad (4)$$

By equating the reaction forces of semi-active system with active system, the semi-active control force can be written as

$$\mathbf{u}_{semi}(t) = \mathbf{u}_{semi}(t) = -(c(t) - c_0) \dot{\mathbf{z}}(t) \quad (5)$$

where now u_{semi} becomes a scalar function, u_{semi} . $z(t)$ = relative displacement across the variable damper, and c_0 = constant optimal passive damping.

Inequality Constraint on Damper Parameter

It is assumed that the variable damping coefficient $c(t)$ can be varied in a range,

$$c_{\min} \leq c(t) \leq c_{\max} \quad (6)$$

To find the optimal variation of damping coefficient which minimizes the performance measure Eq.(3) subjected to the inequality constraints Eq.(6), we define the Lagrangian (Bryson and Ho 1975)

$$H(c(t), \lambda_1, \lambda_2) = \frac{1}{2} L(c(t), k) + \lambda_1 f_1(c(t)) + \lambda_2 f_2(c(t)) \quad (7)$$

where $f_1(c(t)) = c(t) - c_{\max} \leq 0$ and $f_2(c(t)) = c_{\min} - c(t) \leq 0$ are the imposed inequality constraints and $\lambda_1, \lambda_2 \geq 0$ are the Lagrange multipliers.

The necessary condition for a minimum becomes

$$\frac{\partial H}{\partial c} = \frac{1}{2} \frac{\partial L}{\partial c} + \lambda_1 \frac{\partial f_1}{\partial c} + \lambda_2 \frac{\partial f_2}{\partial c} = 0 \quad (8)$$

By substituting Eq.(5) into Eq.(4) and then into Eq.(8), we get

$$c_{opt}(t) = \frac{\lambda_2 - \lambda_1}{\dot{z}(t)^2} + \frac{u_{opt}(t)}{\dot{z}(t)} + c_0 \quad (9)$$

More precisely, we have an optimal variation of damping as

$$c_{opt}(t) = \begin{cases} c_{\max} , & \text{if } c_d(t) \geq c_{\max} \\ c_d(t) , & \text{if } c_{\min} \leq c_d(t) \leq c_{\max} \\ c_{\min} , & \text{if } c_d(t) \leq c_{\min} \end{cases} \quad (10)$$

$$\text{where } c_d(t) = u_{opt}(t) / \dot{z}(t) + c_0 \quad \text{for } \dot{z}(t) \neq 0 \quad (11)$$

and the sufficient conditions for minimum is ensured by Eq.(8) and $\frac{\partial^2 H}{\partial c^2} = (\dot{z}(t))^2 \geq 0$.

Vibration Control of a Single-degree-of-freedom System

Problem Formulation and System Properties

In the following numerical examples, a SAMD is employed to reduce the vibration of the main structure which is model by a SDOF system subjected to an harmonic excitation $F(t)$ as shown in figure 2. The equations govern the vibration of the system are

$$(m_1 + m_2) \ddot{x}_1(t) + c_1 \dot{x}_1(t) + k_1 x_1(t) = F(t) - m_2 \ddot{z}(t) \quad (12)$$

$$\text{and } m_2 \ddot{z}(t) + c_2(t) \dot{z}(t) + k_2 z(t) = -m_2 \ddot{x}_1(t) \quad (13)$$

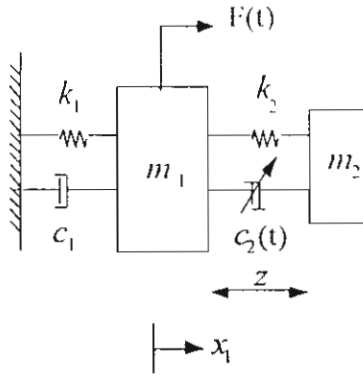


Figure 2. Structure-SAMD Model

where the time-dependent damping of the SAMD, $c(t)$ is computed according to the control law derived in previous section.

Throughout the simulations, the main structure is assumed to have the natural frequency of 3.0 Hz and the damping ratio of 2.0 %. While the mass dampers are assumed to have the mass ratio of 3.0 %. The passive parameters of the TMD are optimized according to the excitation (Den Hartog 1956). Table (1) lists all the values of the system properties, which are used for the numerical model.

Table 1. System Properties

(i) main structure	$m_1 = 1 \text{ kg}$	$k_1 = 355.30 \text{ N/m}$	$\xi_1 = 2.00 \%$
	$\omega_1 = \sqrt{\frac{k_1}{m_1}} = 18.85 \text{ rad/s } (f_1 = 3.0 \text{ Hz})$		
(ii) SAMD	$m_2 = 0.03 \text{ kg}$	$k_2 = 10.05 \text{ N/m}$	
	$c_{2\min} = 0, \quad c_{2\max} = 3.403 \text{ N-s/m } (\xi_2 = 300\%)$		

Response under Harmonic Excitation

It is known that for a nonlinear system the response magnification factor may depend on the excitation or the system response may not be even periodic. Nevertheless, it is demonstrated that (1) the response of the eqs.(12) and (13) to periodic excitation is

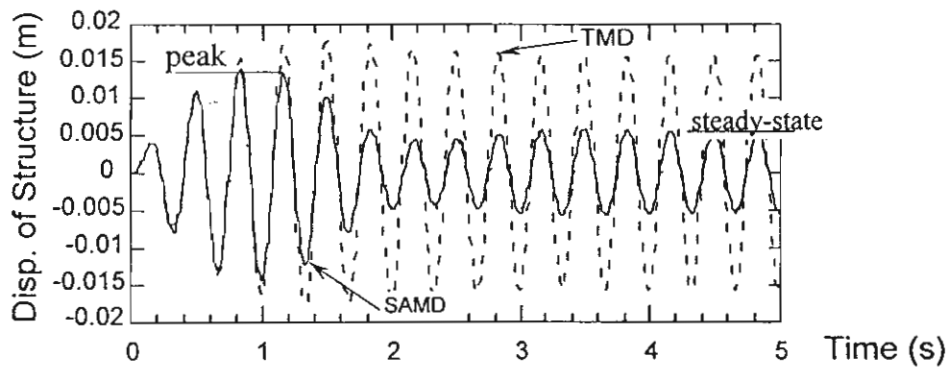


Figure 3. Typical Harmonic Responses under TMD and SAMD

periodic with the same period as excitation, and (2) the amplitude ratio is independent of excitation amplitude (Hac and Youn 1992). Therefore the following comparison of the control performance in frequency domain is meaningful.

Figure (3) shows the typical time histories of the structure subjected to harmonic excitation under TMD and SAMD. Assuming that the system starts from rest, the steady-state amplitude and the peak amplitude are defined as shown in this figure for SAMD case.

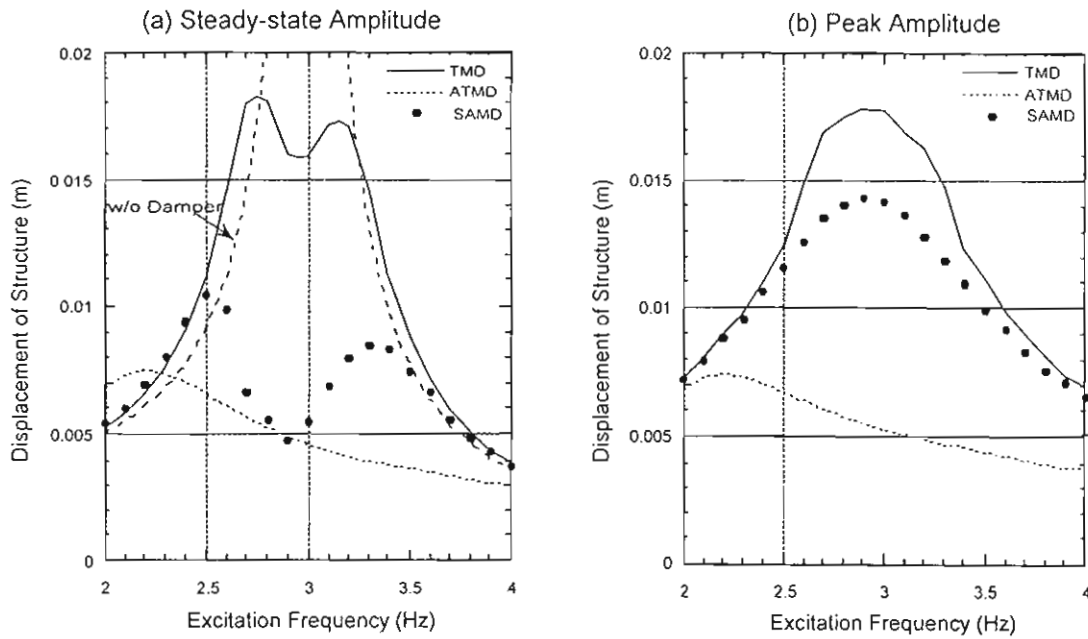


Figure 4. Frequency Responses (a) Steady-state Amplitude and (b) Peak Amplitude.

Figure (4a) plots the steady-state amplitudes of the structure controlled with TMD, ATMD, and SAMD, within the frequency range of 2.0-4.0 Hz. The figure indicates that significant improvement of steady-state amplitude reduction of the structure, comparing to TMD, can be achieved via SAMD. Beside the amplitude reduction, the figure also reveals that SAMD has broader frequency band of suppression. More precisely, the suppression band of SAMD becomes about 2 times broader than that of TMD. This is equivalent to the increasing of mass of TMD by 300 percents. Figure (4b) plots the peak amplitudes of the structure within the same frequency range. It is found that SAMD markedly improves the peak amplitude response of structure over TMD case especially around the resonant frequency. It is clearly from these two figures that, in both cases, ATMD exhibits highest control performance. However, its implementation is known to be very costly.

Conclusion

An optimal control law which minimizes the quadratic performance index for a semi-active mass damper is formulated and applied to control the vibration of SDOF-structure. Through the computer simulation, the control performance of the semi-active mass damper is studied and compared with that of TMD under harmonic excitation. The obtained results reveal that the proposed SAMD can significantly reduce the peak response and also effectively suppress the steady-state response for broader band of excitation frequencies. In particular, the latter improvement is equivalent to the addition of mass about 300% on TMD. This is often unfeasible for passive system implementation.

Generally, simulation results indicate that the semi-active mass damper which is usually cheaper than the active damper system offers a substantially better performance than the passive TMD. Consequently, it represents a very attractive alternative for vibration control of structures subjected to harmonic-like disturbances.

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ENHANCEMENT OF STRUCTURAL PERFORMANCE FOR MODERATE SEISMIC RISK REGIONS

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SYNOPSIS

This paper reports recent research works carried out at the Earthquake Engineering and Vibration Research Laboratory of Chulalongkorn University (CU-EVR) which aim at enhancing the performance of buildings in low to moderate seismic risk regions, which, incidentally, cover about 80 percent of the land on earth. First, enhancement of ductility performance of moderately confined concrete tied columns with hook-clips is presented. Then, the advantage of tuned mass dampers (TMD) in reducing damage of buildings is investigated under moderate ground shaking caused by long distance earthquakes resembling the (scaled) 1985 Mexico City (SCT) or the 1995 Bangkok ground motions.

DUCTILITY ENHANCEMENT OF MODERATELY CONFINED CONCRETE TIED COLUMNS WITH HOOK-CLIPS

Introduction

The deficiency of 90-deg hook ties in columns was witnessed in past earthquakes in bridges, in reinforced concrete buildings and in laboratories.¹⁻⁴ Despite their poor performance, 90-deg hook ties are still used extensively worldwide in low to moderate seismic risk regions because of the ease of their placement compared with the 135-deg hooks. Ninety-degree hook ties are even more appealing in developing countries where laying of reinforcing bars is quite commonly not practiced to an acceptable standard of precision, making it extremely difficult to put 135-deg hook ties in place. Recently, Lukkunaprasit⁵ introduced a simple device called "hook-clip" to be clipped onto the conventional 90-deg hook ties or crossties at the sites. Experimental tests on axially loaded short columns revealed that the performance of r.c. tied columns with 90-deg hooks and hook-clips was comparable to that of columns with 135-deg hook ties.

In view of the importance of vertical load resistance members, it is extremely desirable to have sufficient inherent ductility in columns (to cope with very rare earthquakes) even in low seismic risk zones. It is also important to improve the performance of ACI crossties to ensure their confinement effectiveness, which is essential in areas of high seismic risk. This research investigates the effectiveness of hook-clips in improving the performance of conventional 90-deg hook ties and ACI crossties in columns for a moderate level of ductility. A method is also proposed for predicting the expected displacement ductility factor in terms of the effective confinement steel ratio, applicable over a wide range of ductilities.

Experimental program

Test specimens

Five column specimens 400 mm by 400 mm in cross section and 1500 mm in height reinforced with 16-20 mm diameter deformed bars served as test specimens. The specimens were moderately confined with transverse reinforcement consisting of 1 hoop tie and 2 ACI crossties at each level, with an amount about half of that required by the ACI Code for high seismicity areas. The ties were supplied with either 90-deg or 135-deg hooks, depending on specimens. The reinforcement detailing

was in accordance with the non-seismic detailing provisions in the ACI Code. Table 1 lists the relevant data of the test specimens. Hook-clips (Fig. 1) were supplied in specimens CFL90/0.30 and CFL90/0.37 to investigate their effectiveness in enhancing the column ductility when compared with CF90/0.30, CF135/0.30 and CF135/0.37. The notations 0.30 and 0.37 designate the ratio of the axial compression to the strength based on the concrete strength on the day of testing f'_c and the gross sectional area of the column.

Table 1 - Details of test specimens

Specimen	Concrete Strength f'_c (MPa)	dimensions			Longitudinal Reinforcement		Transverse Reinforcement				Hook Configuration	$P/f'_c A_g$
		width (mm)	depth (mm)	height (mm)	ρ_l (%)	f_y (MPa)	Diameter (mm)	Spacing (mm)	f_{yh} (MPa)	A_{sh}/s (%)		
CF90/0.30	38.9	398	397	1500	3.14	471	9	120	305	0.453	90° + ACI cross-ties; no clips	0.30
CF135/0.30	35.7	398	396	1490	3.14	471	9	120	305	0.453	135° + ACI cross-ties; no clips	0.30
CFL90/0.30	31.7	398	398	1500	3.14	471	9	120	306	0.453	90° + ACI cross-ties with clips	0.30
CF135/0.37	30.5	399	397	1500	3.14	475	9	120	318	0.453	135° + ACI cross-ties; no clips	0.37
CFL90/0.37	32.4	398	397	1500	3.14	471	9	120	297	0.453	90° + ACI cross-ties with clips	0.37

Test setup and instrumentation

Fig. 2 shows the schematic diagram of the test setup. The axial load on each specimen was applied by means of a hydraulic jack bearing against the column top and a load transfer girder sitting on top of the jack. Linear variable differential transformers (LVDTs) were employed to measure the lateral displacements of the column along its height. Selected ties in the plastic hinge zone were instrumented with electrical resistance strain gages. The 1000 kN hydraulic actuator for horizontal load application was fitted with a load cell. Signals from the load cell, LVDTs and strain gages were connected to a computerized data acquisition system.

Testing procedure

The testing was carried out following the general procedure proposed by Watson and Park⁶. After the application of the specified axial load, the lateral force was load controlled to ± 75 percent of the theoretical lateral yield value computed on the basis of the ACI Code without any strength reduction. Subsequently each specimen was subjected to displacement controlled cyclic loading, starting from the displacement ductility factor μ_Δ of 1. The displacement ductility level was incremented at an interval of 1, in general, with two cycles of loading performed for each ductility level until failure.

Test results

Lateral load-displacement hysteretic response

The lateral load-displacement hysteretic responses for the test specimens are shown in Fig. 3-4. The curves clearly indicate flexural dominated characteristics. It is interesting that specimens CF135/0.30 and CFL90/0.30 experienced stable hysteresis loops up to ductility level 3.0, with similar general characteristics. While the specimen with 135-deg hooks and (unclipped) ACI cross-ties suffered sharp decrease in lateral load resistance during the next (and last) loading cycle to ductility level 4.0, the specimen with hook-clips exhibited a stable hysteresis loop in the same cycle, with little strength degradation. The latter was able to sustain 2 complete cycles at this ductility

level, although significant degradation in strength and stiffness occurred in the second cycle at $\mu_{\Delta} = 4.0$.

The sudden drop in lateral load resistance was even more pronounced in specimen CF90/0.30 when it was being pushed to $3\Delta_y$. This was caused by the opening of the 90-deg hook of the ACI crosstie in the second tie set above the base, when the longitudinal bars buckled.

Specimen CFL90/0.37 exhibited a more stable hysteretic response than CF135/0.37, with much less strength degradation. Prior to unloading from the second cycle at $\mu_{\Delta} = 3$, specimen CF135/0.37 rapidly lost its load capacity (by more than 45 percent) due to buckling of the vertical bars at a displacement of $+2.54\Delta_y$. The specimen with hook-clips, on the other hand, could still carry 88 percent of the peak load, and even sustained one complete cycle at $\mu_{\Delta} = 3.5$ with a remarkable capacity before final failure.

Effectiveness of hook-clips

From the following observations, it is evident that the hook-clips were effective in improving the performance of the 90-deg hooks in the hoop ties and ACI crossties:

a) In contrast to CF90/0.30 with conventional 90-deg hooks, the clips in the CFL90/0.30 specimen were able to prevent premature opening of the 90-deg hooks, leading to effective restraint of the vertical bars at the tie positions, and resulting in the vertical bars buckling by about half the buckling length of those without clips as depicted in Fig. 5. The shorter buckling length allowed the specimen to sustain a higher load at the same deformation.

b) The ductility performance of the specimen with 90-deg hook ties and hook-clips far excelled that of the specimen without hook-clips, by a factor of 1.8 in terms of displacement ductility attained at failure (see Table 2). In fact, the former even performed better than the CF135/0.30 which was confined with 135-deg ties and ACI crossties (without hook-clips).

c) The crossties with hook-clips were able to develop larger strains than those without clips. From the strain gage readings it was found that the most severely strained crosstie in specimen CFL90/0.37 developed a maximum strain of 1.5 times that in CF135/0.37.

Table 2 - Test results

Specimen	Lateral reinforcement ratio $A_{st}/(sh_c)$			Δ_y (mm)	Δ_{max} (mm)	$\mu_{\Delta,exp}$	Failure mode
	Provided	ACI Code	$A_{st,prov}/A_{st,ACI}$				
CF 90/0.30	0.0045	0.0115	0.39	143	284	2.0	Flexure
CF 135/0.30	0.0045	0.0105	0.43	142	426	3.0	Flexure
CFL 90/0.30	0.0045	0.0093	0.49	150	600	3.7	Flexure
CF 135/0.37	0.0045	0.0086	0.52	142	426	2.5	Flexure
CFL 90/0.37	0.0045	0.0088	0.46	132	396	3.0	Flexure

Confinement steel for expected ductility factor

It has been widely recognized that, while the ACI Code provisions for the design of confinement steel for earthquake-resistant design of columns are conservative for low compression loads, they may not be sufficient for high levels of compression. Furthermore, the code provisions are specified for high seismic risk regions, and no recommendations on the amount of confinement steel are given for moderate levels of ductilities suitable for areas of moderate seismic risk, although some

requirements are specified on tie spacing limits in the critical end regions around beam-column joints.

To cover a wide range of confinement and axial load levels, as well as to take account of tie configurations, the test results obtained from this study together with existing experimental results carried out by various researchers on reinforced concrete square and rectangular columns were used in a regression analysis to obtain a relationship between the displacement ductility factor and the 'effective confinement reinforcement ratio' ρ_A defined as

$$\rho_A = \frac{A_{sh} k'_e}{A_{sh,ACI} Y_p} \quad (1)$$

where k'_e is the confinement effectiveness coefficient⁷ normalized with respect to 0.75, which is the upper bound value assumed by the ACI Code for the efficiency of rectilinear ties in comparison with that of closely-spaced spirals; $A_{sh,ACI}$ is the minimum total cross-sectional area of rectangular hoops and crossties as specified by the ACI Code; and Y_p is the axial load parameter which takes into account the effect of the axial load level⁸.

A simple design chart is proposed by Lukkunaprasit⁹ shown as the dotted line in Fig. 6., which sets a practical lower bound for the ductility performance of columns under normal compression level, as well as a suggested allowable minimum effective confinement steel ratio and an upper bound for the ductility factor that may be attained in practice.

INELASTIC BUILDINGS WITH TUNED MASS DAMPERS UNDER LONG DISTANCE EARTHQUAKES

The problem

Many big cities in moderate seismic risk regions, such as Bangkok and Singapore, are known to be situated at sufficiently long distances from major active faults. The ground motions induced by long distance earthquakes, amplified by the soft soil strata present in many locations, may lead to resonance-like motion which poses a threat to medium to high-rise buildings. However, the harmonic-like ground motion from long distance earthquakes coupled with long excitation duration leads to a favorable condition for the TMDs in vibration control of buildings. While several studies exist¹⁰⁻¹³ which were directed toward high intensity earthquake motions, little attention has been given to moderate ones, for which TMD should be of advantage in reducing damage in buildings which would otherwise be heavily damaged if the TMDs were not equipped. Furthermore, improved performance of buildings is also of increasing importance in view of the current trend toward performance-based design which, depending on the performance objectives, might call for economical restorability under the damage control limit state.

In this paper, the effectiveness of TMD in reducing damage in buildings undergoing inelastic deformation under ground shaking pertinent to moderate seismic zones is demonstrated through numerical examples of 5-storey shear buildings undergoing elastic-perfectly plastic deformations. Linear TMDs were employed by virtue of their simplicity and robustness. In each building, the bottom storey yield strength was taken as 1/4 of the elastic base shear which would result if the building was subjected to the scaled Mexico excitation. The damping ratio of the building was taken as 2% for each vibration mode. The TMD damping ratio and tuning frequency were based on the Den Hartog method¹⁴, with the generalized mass ratio taken as 6%.

The excitations considered were the 1985 Mexico city (SCT record) and the 1995 Bangkok accelerograms, scaled to the value of 0.10g to represent moderate ground shaking (Fig. 7). The response spectra of the ground motions are shown in Fig. 8.

The details of the study are given in a paper by Lukkunaprasit and Wanitkorkul¹⁵. A brief summary of the main findings is described herein.

TMD performance with variation of building natural period

As evident in Fig. 9, the TMD performance is close to optimum when the building period is in the neighborhood of the predominant period of the Mexico ground motion. Elsewhere, the detuning effect will be more pronounced leading to deterioration in TMD performance. It may be observed that over the building period range of 1.8-2.8 sec, the reduction in the dissipated energy is 40% or more even though there is an insignificant reduction in the peak displacement response (Fig. 10). As an example, for the idealized 5-DOF building whose fundamental period was 2.0 sec under the scaled Mexico earthquake, the peak displacement for the building with TMD was reduced by a small amount of 12% from that of the uncontrolled building, whereas the number of yielding cycles in the ground floor columns was reduced significantly by about 70% with the aid of TMD (Fig. 11). This could be best quantified by means of the accumulated hysteretic energy absorption ratio, which was found to be 0.20. In other words, the hysteretic energy dissipation in the building with TMD was only 20% of that in the uncontrolled one. For other cases, the number of yielding excursions in the bottom storey is reduced by more than 40% in general. The results clearly indicate that TMD can be effective in reducing the inelastic deformation, with the consequence of less damage, for buildings in the 1.8-2.8 sec period range when they are subjected to motions with characteristics similar to the Mexico City (SCT) or Bangkok earthquakes.

Damage reduction by TMD for moderate ground shaking

As mentioned earlier, the earthquakes considered in this study were intended to represent rare events in moderate seismic risk regions. The buildings without TMD, though capable of withstanding the ground shaking, would suffer significant damage. Depending on the design for ductility as well as characteristics of the hysteresis loops, a value of 15 or more may be the ultimate limit of the hysteretic energy normalized with respect to $f_y \Delta_{y1}$ of the bottom storey. Taking a value of 15 as the limit, for instance, the 2 sec period building without TMD would be on the verge of collapse under the scaled Mexico ground motion with PGA of 0.10g. However, with the contribution of TMD, the critical columns in the building-TMD system are now called upon to dissipate only 20% of the hysteretic energy of the system without TMD, while the ductility demand is also reduced slightly. Consequently, the building with TMD would survive the earthquake with much less damage. More details are given by Lukkunaprasit and Wanitkorkul¹⁵.

Conclusions

Two important problems relevant to many cities in this region are addressed. Firstly, a significant improvement of the ductility performance of non-ductile reinforced concrete tied columns prevalent in current design practice can be achieved with the use of hook-clips, which can be easily and practically placed on site to restrain the opening of the 90-deg hook ties and ACI crossties. In fact, the overall performance of the specimens with hook-clips under a moderate ductility demand was even superior to that of columns confined with 135-deg hook ties and conventional ACI crossties. The effective restraint of the 90-deg ends of crossties by hook-clips led to a significant improvement of ACI crossties, which should be beneficial even in regions of high seismicity. Secondly, with the aid of TMDs, it is possible to achieve a reduction of damage in buildings in the 1.8-2.8 s range

caused by moderate ground shaking induced by long distance earthquakes resembling the 1985 Mexico City (SCT) or the Bangkok ground motion.

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Fig. 1 - Hook-clips engaging 90°
hoop ties and crossies

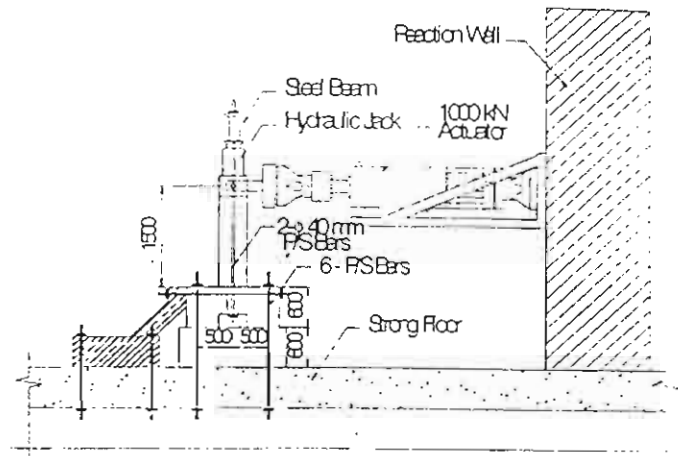


Fig. 2 - Test setup

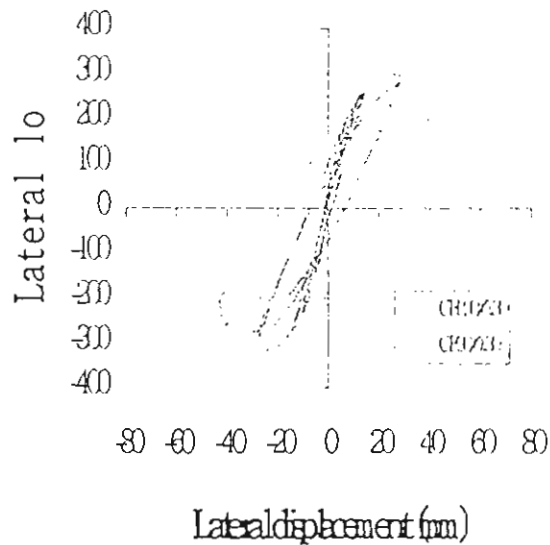


Fig. 3 - Lateral load-displacement hysteresis
for specimens CF90/0.30 and CFL90/0.30

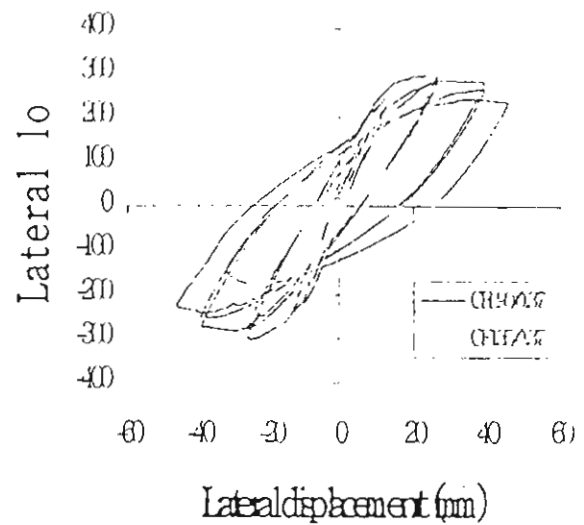


Fig. 4 - Lateral load-displacement hysteresis
for specimens CF135/0.37 and CFL90/0.37



(a) over 2 tie spacings in CF90/0.30



(b) over single tie spacing in CFL90/0.30

Fig. 5 - Buckling of vertical bars

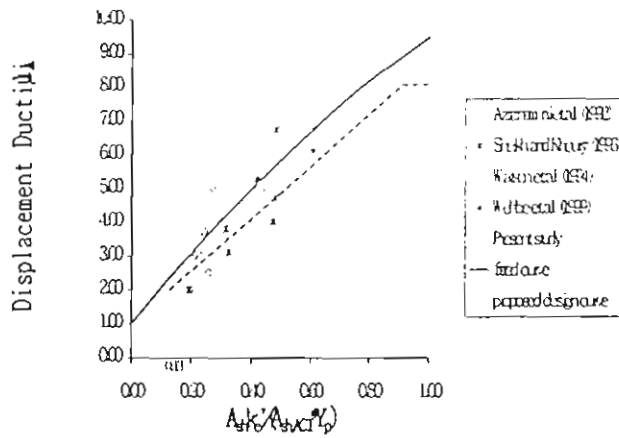


Fig.6 - Ductility performance as a function of

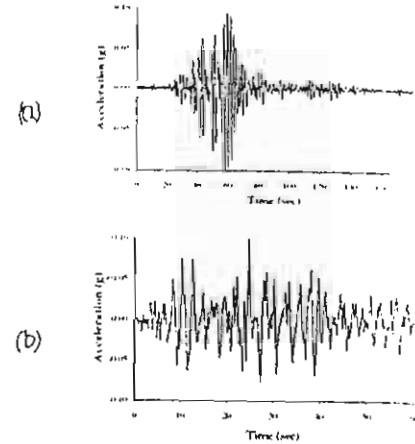


Fig. 7 - Scaled ground motion records:
effective confinement reinforcement ratio
(a) 1985 Mexico city (SCT);
(b) 1995 Bangkok

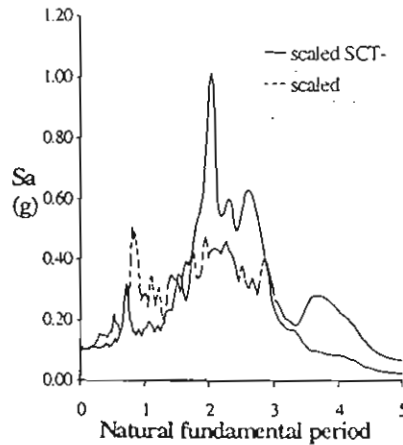


Fig. 8. - Response spectra of scaled Mexico
and Bangkok ground motions
(0.10g PGA, 2% critical damping)

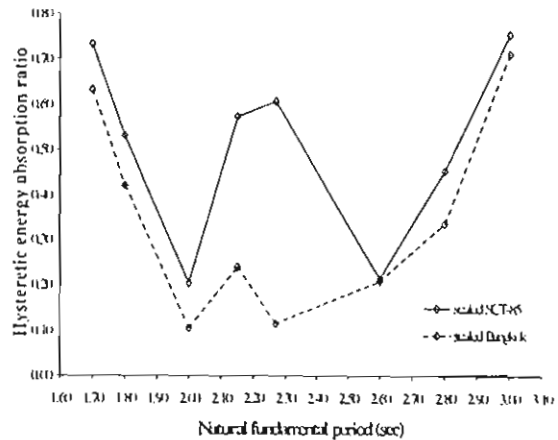


Fig. 9 Variation of hysteretic energy
absorption ratio with natural period
of building

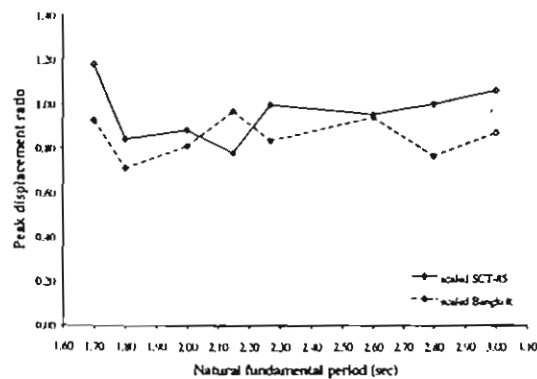


Fig. 10 - Variation of peak displacement ratio with
atural period of building

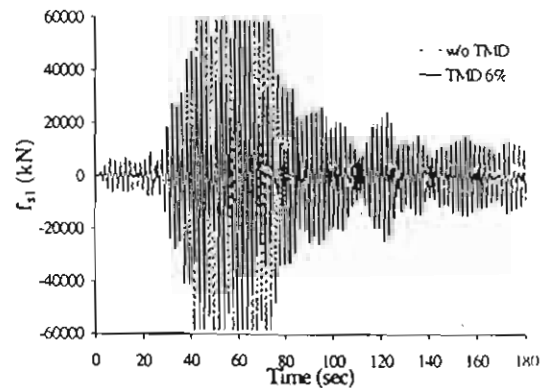


Fig. 11 - Base shear response of elasto-
plastic shear frame with $T_1=2.0\text{sec}$ to
scaled Mexico (SCT-85) ground motion



AN INNOVATIVE HOOK-CLIP FOR PERFORMANCE IMPROVEMENT OF REINFORCED CONCRETE TIED COLUMNS

Panitan LUKKUNAPRASIT¹

SUMMARY

This paper presents a simple-to-place device, called "hook-clip", which can be used to hold the legs of the conventional 90° hook ties or crossties in reinforced concrete columns at the sites, so as to effectively improve their performance. The hook-clip invented is to prevent premature opening of the legs of the hooks. Static compression tests on a number of specimens clearly demonstrated the effectiveness of the clips in restraining the vertical reinforcing bars at the tie positions, resulting in the bars buckling in about half the buckling length of those in specimens without hook-clips and hence higher sustained load at the same level of strain. At a large axial strain of 0.04, columns with hook-clips were able to sustain a loading comparable to that of columns confined with 135° hook ties. Preliminary test results of a medium size column under cyclic loading are also presented.

INTRODUCTION

Although extensive studies of reinforced concrete columns confined with 135° hook ties have been carried out [e.g. Kent and Park 1971; Mander et al. 1988; Park et al. 1982; Sheikh and Uzumeri 1980], very little work has been focused on the performance of 90° hook ties, inspite of the fact that crossties with 135° hook at one end and 90° hook at the other are permitted by ACI Code [1995] even in areas of high seismic risk. Razvi and Saatcioglu [1989] tested 2 specimens with 90° hook ties and their results indicated that they were inferior to columns confined by 135° hooks at axial strains in excess of about 0.015. Shiekh and Yeh [1990] investigated the behavior of tied columns with different reinforcement and tie configurations under medium to high axial load levels and flexure. Crossties with 90° hooks were reported to cause brittle failure and to be harmful rather than beneficial, especially at high axial loads. Lynn et al. [1996] tested eight full-scale reinforced concrete columns having details widely practiced before mid 1970's in U.S.A. and including 90° hook tie details among others. Cyclic load-displacement curves were obtained for light and moderate level axial loads. Poor performance of 90° hook ties was evident, leading to rapid loss of gravity load resistance.

The deficiency of 90° hook ties in columns was witnessed in past earthquakes in bridges [e.g. Seible et al. 1995], reinforced concrete buildings [EQE. 1995], and steel reinforced concrete structures [Azizinamini et al. 1997]. Despite their poor performance, 90° hook ties are still used extensively worldwide in low to moderate seismic risk regions because of the ease of their placement, when compared with the 135° hooks. Ninety-degree hook ties are even more appealing in developing countries where laying of reinforcing bars is quite commonly not practiced to an acceptable standard of precision, making it extremely difficult to put 135° hook ties in place. In view of the importance of vertical load resistance members, it is extremely desirable to have sufficient inherent ductility (to cope with very rare earthquakes) even in low seismic risk zones. It is also equally important to improve the performance of conventional crossties with 90° hooks to ensure their confinement effectiveness, which is essential in areas of high seismic risk. This paper presents a simple-to-place device called "hook-clip" to be clipped onto the conventional 90° hook ties or crossties at the sites so as to effectively improve their

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(without hook-clips). Normal weight concrete with a maximum aggregate size of 20 mm and a slump of 75 mm-125 mm was used. Typical stress-strain curves of the reinforcing bars used are shown in Fig. 4.

Each test specimen was instrumented with LVDTs on four faces with nominal gauge lengths of 200 mm. The columns were loaded at a slow strain rate using a 5000-kN hydraulic compression testing machine.

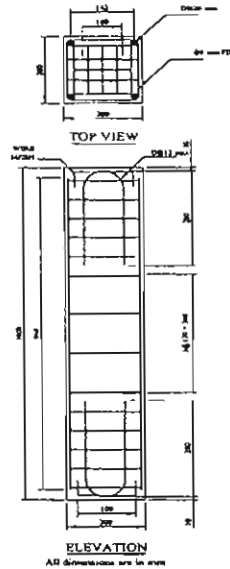


Figure 3: Detail of typical reinforced concrete column specimen (hook-clips not shown)

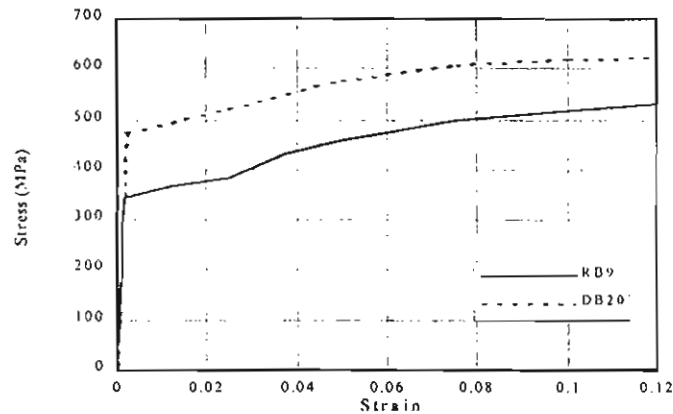


Figure 4: Stress-strain relationships of reinforcing bars

Table 1: Details of column specimens

Specimen	Concrete strength MPa	Longitudinal reinforcement			Lateral ties			Hook angle degree
		Main bar	f_y MPa	ρ_{gross} (%)	Diameter mm	f_{yh} MPa	ρ_h (%)	
C90-1	31	4DB20mm	475	3.14	8.9	350	1.2	90
C90-2	31	4DB20mm	475	3.14	8.9	350	1.2	90
C90-3	35	4DB20mm	475	3.14	8.9	350	1.2	90
CL90-1	31	4DB20mm	475	3.14	8.9	350	1.2	90 with clips
CL90-2	31	4DB20mm	475	3.14	8.9	350	1.2	90 with clips
CL90-3	35	4DB20mm	475	3.14	8.9	350	1.2	90 with clips
C135-1	31	4DB20mm	475	3.14	8.9	350	1.2	135
C135-2	31	4DB20mm	475	3.14	8.9	350	1.2	135
C135-3	35	4DB20mm	475	3.14	8.9	350	1.2	135

Notes: f_y = yield strength of longitudinal reinforcement ; f_{yh} = yield strength of ties
 ρ_{gross} = ratio of longitudinal steel area to gross column area
 ρ_h = ratio of volume of tie steel to volume of concrete core measured to centers of the tie

TEST RESULTS

Effectiveness of hook-clips

As expected, soon after the concrete covers were spalled off, the 90° hook ties which were most severely loaded started to open up under the outward thrust resulting from the lateral movement of the longitudinal bars during buckling, with a rapid decrease in load carrying capacity. This clearly indicated that these ties were not able to

performance. The effectiveness of the hook-clip was examined in comparison with specimens with conventional 90° and 135° hook ties under static concentric compression loading. Preliminary test results of a medium size column under cyclic lateral loading are also included.

THE HOOK-CLIP

To prevent premature opening of the ties, it is envisaged to provide a supplementary tie or "hook-clip" holding the legs of the hooks and embedded in the concrete core. The clip would provide an inward thrust resisting the opening of the ties after the concrete cover spalls off. Fig. 1 shows the details of the clip proposed for binding 9 mm diameter ties or smaller. The hook-clip may be employed to clip the legs of any hoop tie or crosstie with 90° hooks (see Fig. 2). With the clips prefabricated, the clipping operation can be done easily at the site. No welding is needed.

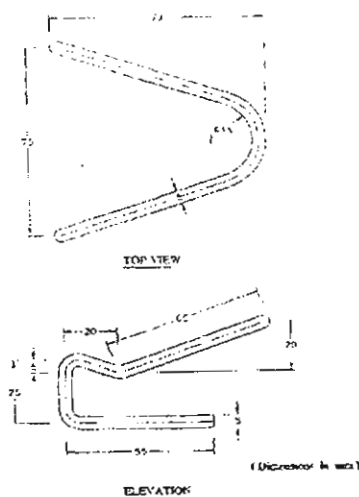


Figure 1: Detail of hook-clip



Figure 2: Hook-clips engaging 90° hoop ties and crossties

TEST SPECIMENS AND TEST SETUP

Small sized column specimens with 200 mm by 200 mm in cross section and 1000 mm in length served as test specimens. Each was reinforced with 4 longitudinal deformed bars of 20 mm (DB20) nominal diameter. The center 360 mm portion of the column was bound by ϕ 9 mm round bars spaced at 120 mm. Close tie spacings of 50 mm were provided in the rest of the column to prevent failure in the end zones. The ties were supplied with hooks (90° or 135° depending on specimens) with an inside radius of 2 times the tie diameter and an extension of 60 mm. The tie spacing of 120 mm is smaller than that stipulated by ACI [1995] which allows 200 mm for low ductility detailing in low to slightly moderate seismic zones. The close spacing was chosen in view of the higher demand on the ties in providing lateral restraint for the longitudinal bars when buckled in shorter unsupported lengths. Consequently, a higher demand was also imposed on the hook-clips in order to prevent the ties from premature opening. The clips were made of 5 mm diameter round bars and embedded 75 mm in the concrete core. The yield strength and modulus of elasticity of the clip steel were 450 MPa and 206,600 MPa, respectively.

It should be noted that large longitudinal bars were used so that at longitudinal bar buckling, a large outward thrust would be exerted on the ties, which would in turn try to pull the hook-clips out of the confined core. Furthermore, the design was meant for regions of low to moderate seismicity, with detailing conforming to ACI Code, which allows for substantially larger spacings of longitudinal and transverse reinforcements than those allowed in higher seismic risk areas.

Three sets of specimens designated C135, C90 and CL90 representing columns with 135°, 90° hook ties, and 90° hook ties with hook-clips, respectively, were tested, each set consisting of ideally identical reinforcement. Table 1 lists the relevant data of the test specimens. Fig. 3 depicts the drawing of a typical column specimen

restrain the longitudinal bars at the tie positions, with the consequence of the vertical bars buckling in approximately twice the tie spacing (see Fig. 5). In contrast to the conventional 90° hooks, the clips in CL90 specimens were able to prevent premature opening of the 90° hooks, leading to effective restraint of the vertical bars at the tie positions, and resulting in the vertical bars buckling in about half the buckling length of those without clips, when buckling commenced, as depicted in Fig. 6. In fact, the clips in all specimens were still holding the hook ends even at sufficiently large strains in excess of 0.040, although the legs of the hooks did slide along the sides of the ties in two specimens.

The overall performance of the specimens with 135° hooks was slightly better than the CL90 series. As expected, the 135° hooks effectively confined the core up to a large axial strain (of about 0.045), after which there was also a tendency for one of the legs to open up. The buckling shape of the longitudinal bars was also similar to the specimens with hook-clips as can be seen in Fig. 7.

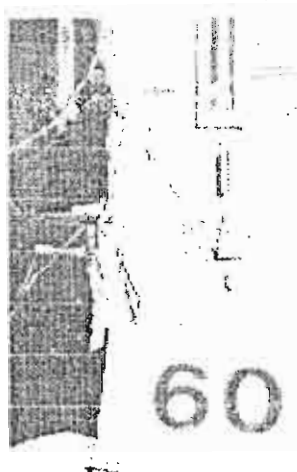


Figure 5: Typical failure mode of column specimen type C90



Figure 6: Typical failure mode of column specimen type CL90

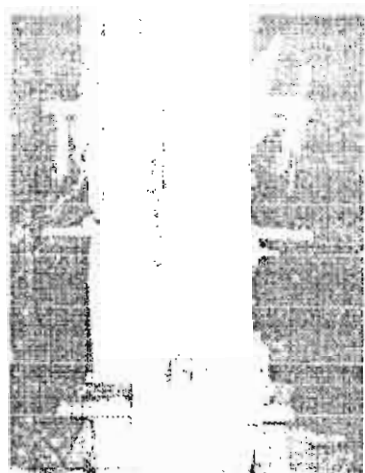


Figure 7: Typical failure mode of column specimen type C135

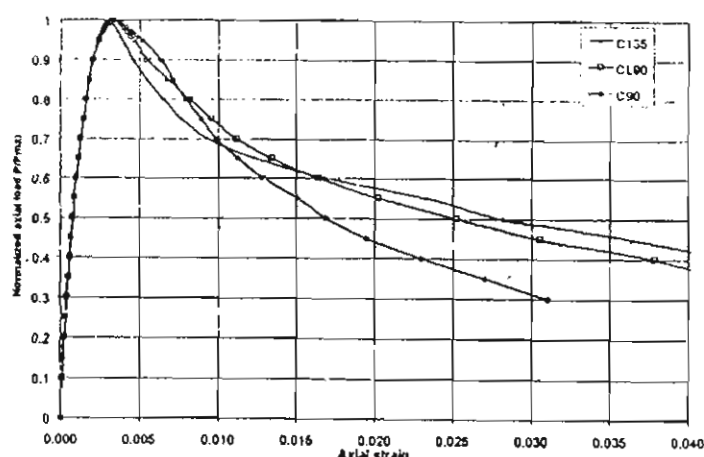


Figure 8: Average normalized load - strain relationships for different tie configurations

Normalized axial load-strain relationships

The peak load for each specimen is tabulated in Table 2. As expected, the peak loads would be practically not influenced by the details of the tie hooks because confinement provided by all hook configurations is essentially equivalent at small strains.

Fig. 8 depicts the average normalized axial load-strain relationships for specimens in each series. Because of some inconsistency in load control the results for the C135-2 specimen were deemed inconsistent and were

therefore discarded. For the 90° hook tied specimens, the load-deformation curve descended rapidly soon after reaching the maximum capacity, indicating loss of confinement due to the opening up of the ties, whereas specimens with hook-clips showed decreasing rate of drop in load carrying capacity. It is evident that the CL90 specimens were much superior to the C90s. The latter practically failed at a strain of about 0.030 while those with hook-clips were still able to carry a significant load of 45% of the peak load at this strain. In fact, the CL90 specimens exhibited load-deformation characteristic quite similar to those confined with 135° ties.

Table 2: Test results - peak loads

Specimen	C90-1	C90-2	C90-3	CL90-1	CL90-2	CL90-3	C135-1	C135-3
Concrete Strength, MPa	31	31	35	31	31	35	31	35
Experimental Peak Load, kN	1805	1940	2000	1795	1910	1980	1830	2020

THEORETICAL AXIAL LOAD-STRAIN CURVES

By assuming a uniform axial strain distribution and suitable stress-strain models for the unconfined concrete shell, the confined concrete core as well as the longitudinal steel bars, one can compute the theoretical axial load-strain relationships based on perfect bond between concrete and steel.

In view of the poor performance of the columns with conventional 90° hooks, and the prevalence of previous work on columns confined with 135° hooks, only the axial load-strain curves for the CL90's are presented. Fig. 9 shows the average test results of CL90-1 and CL90-2 (whose compressive strengths were the same at the time of testing) together with the predicted curves based on the well known Modified Kent and Park [Park et al. 1982] and the Mander [1988] models. The corresponding results for CL90-3 are plotted in Fig. 10.

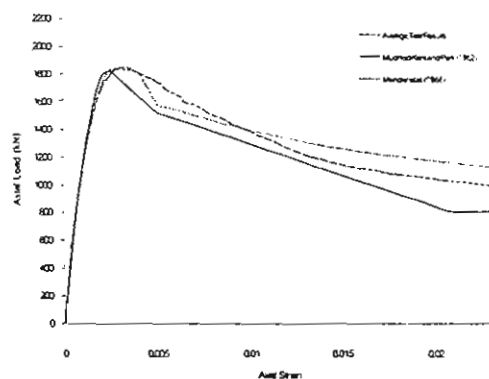


Figure 9: Comparison of theoretical and experimental axial load-strain curves-Column CL90-1 and CL90-2

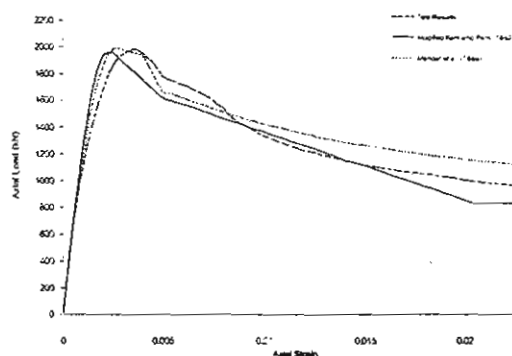


Figure 10: Comparison of theoretical and experimental axial load-strain curves-Column CL90-3

In computing the theoretical values, the strength of the concrete in the specimens was taken as 90% of that obtained from the standard cylinder tests. The spalling strain was assumed to be 0.005. Furthermore, buckling of the longitudinal bars was ignored, and the bars were assumed to have an elastic-perfectly plastic stress-strain relationship. Since the loading capacity of columns with moderate confinement generally drops more than 20% from the peak value when buckling of the vertical bars takes place, which corresponds to an axial strain of about 0.015 in the CL90 series, these assumptions are reasonable for the prediction of theoretical axial load-strain relations in the useful range of the column. It may be observed that, in general, the Mander model gives a better estimate of the axial load carried by the CL90 columns at small strains up to about 0.010. For larger strains in the range of 0.010-0.015, the Mander model overestimates the load capacity (by about 10-14%) while the Modified

Kent and Park model gives a more conservative and closer estimate. Beyond the axial strain of about 0.015, a more refined analytical method is needed to account for buckling of the longitudinal bars and deterioration of the concrete core, which are neglected in this study.

PRELIMINARY CYCLIC LOAD TEST

With the promising performance of hook-clips evidenced in monotonic static load tests, investigation on the behavior of a cantilevered column of medium size (400 mm x 400 mm x 1500 mm) under a moderate level of axial load ($0.30 f_c'$ times the gross sectional area) and cyclic lateral loading was carried out. The reinforcement consisted of 16-20 mm diameter longitudinal bars bound by sets of one 9 mm diameter hoop tie with two cross-ties spaced at 120 mm spacings. Hook-clips were used to engage the 90° hooks of the ties and cross-ties (Fig. 2). Preliminary test results did confirm the effectiveness of the hook-clips in restraining the vertical bars at the tie positions after the specimen was subjected to 7 cycles of lateral load reversals with a maximum displacement ductility factor of about 4. As is evident from Fig. 11, the hook-clips successfully tied the 90° hooks, resulting in the buckling shape of the vertical bars in resemblance with that in the monotonic compression test. An extensive test program is currently underway to investigate the cyclic performance of hook-clips.



Figure 11: (a) Cantilevered column under cyclic load test; (b) Effectiveness of hook-clip in restraining vertical bars at tie positions

CONCLUSIONS

The hook-clips invented are effective in improving the performance of axially loaded columns confined with 90° hook ties, as evidenced from the buckling lengths of the longitudinal bars in specimens with hook-clips being about half of those in specimens without hook-clips, when bar buckling commenced. The latter specimens practically failed at a strain of about 0.030 while a significant portion of the peak load could still be carried by the specimens with clips by virtue of the better confinement enhanced by the hook-clips and the higher buckling strength of the vertical bars. In fact, the overall performance of the specimens with hook-clips was comparable to that of columns confined with 135° hook ties. The theoretical axial load-strain relationships can be reasonably predicted by using the Mander concrete model for strains prior to incipient buckling.

Conventional 90° hook tied columns do not possess sufficient ductility for a safe environment in the event of a rare earthquake. The effectiveness of hook-clips in improving the performance of tied columns can contribute to earthquake risk reduction in low to moderate seismic risk zones, especially in developing countries where changes in construction habit cannot be easily effected. The easy-to-place hook-clips would provide an attractive alternative solution, subject to further extensive research work on their cyclic performance.

ACKNOWLEDGMENTS

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ABSTRACT

Although reinforced concrete columns confined with 90° hook ties have demonstrated very poor performance both in past earthquakes and in laboratories, they are still used extensively worldwide in low to moderate seismic risk regions because of the ease of their placement, when compared with the 135° hooks. The problem is even more pronounced in developing countries where laying of reinforcing bars is quite commonly not practiced to an acceptable standard of precision, making it extremely difficult to put 135° hook ties in place. Moreover, crossties with 135° hook at one end and 90° hook at the other are permitted by ACI Code even in areas of high seismic risk. In view of the importance of vertical load resistance members, it is extremely desirable to have sufficient inherent ductility (to cope with very rare earthquakes) even in low seismic risk zones. It is also equally important to improve the performance of conventional crossties with 90° hooks to ensure their confinement effectiveness, which is essential in areas of high seismic risk. This research paper presents a simple-to-place device called "hook-clip" to be clipped onto the conventional 90° hook ties or crossties at the sites, so as to effectively improve their performance. The effectiveness of the hook-clip was examined in comparison with specimens with conventional 90° and 135° hook ties under static compression.

Nine short columns with 200 mm x 200 mm in cross section and reinforced with non-seismic detailing were tested to failure. The tie configurations included were conventional 135° -, 90° - hooks and 90° - hooks with hook-clips. The tie spacing and vertical reinforcement were designed in such a way that a large strength demand was imposed on the ties in providing lateral restraint for the longitudinal bars, and hence a large demand was also imposed on the hook clips.

The hook-clips invented are effective in improving the performance of axially loaded columns confined with 90° hook ties, as evidenced from the buckling lengths of the longitudinal bars in the specimens with hook-clips being about half of those in specimens without hook-clips. The latter practically failed at a strain of about 0.030 while a significant portion of the peak load could still be carried by the specimens with clips by virtue of the better confinement enhanced by the hook-clips and the higher buckling strength of the vertical bars. In fact, the overall performance of the specimens with hook-clips was comparable to that of columns confined with 135° hook ties. A preliminary test on cyclic behavior of a cantilevered column specimen of medium size also confirmed the effectiveness of the hook-clips in restraining the vertical bars at the tie positions. The effectiveness of hook-clips in improving the performance of tied columns can contribute to earthquake risk reduction in low to moderate seismic risk zones, especially in developing countries, where simplicity is an important factor in construction practice.

DEVELOPMENT OF REINFORCEMENT DETAILS TO IMPROVE THE CYCLIC RESPONSE OF SLENDER STRUCTURAL WALLS

Chadchart SITTIPUNT¹ And Sharon L WOOD²

SUMMARY

A series of four reinforced concrete walls were tested to failure to evaluate the influence of diagonal web reinforcement on the hysteretic response. Two walls contained conventional horizontal and vertical web reinforcement and two walls contained inclined reinforcement. Reinforcement details were representative of construction practice in regions of low to moderate seismic risk. A single layer of web reinforcement was used and the transverse reinforcement in the boundary elements did not confine the concrete core.

Both walls with conventional web reinforcement failed due to web crushing. Pinched shapes characterized the hysteresis curves for top displacement and shear distortion near the base. In contrast, the walls with diagonal reinforcement displayed rounded hysteresis curves, and failed due to crushing of the boundary elements. The choice of web reinforcement did not have a significant influence on the maximum lateral load resisted by the walls, but measured crack widths were less, and more energy was dissipated by the walls with diagonal reinforcement during loading cycles to comparable levels of displacement.

INTRODUCTION

Fundamental studies of the behavior of slender reinforced concrete structural walls were conducted during the 1970s at the Portland Cement Association [Oesterle, et al, 1976, and Oesterle, et al, 1979]. Experimental parameters included the amounts of longitudinal, web, and confinement reinforcement. The results of these tests were used to establish reinforcement requirements in current building codes [ACI 318, 1995], and were later interpreted to investigate the influence of the amount of reinforcement on wall behavior [Wood, 1991]. Subsequent analytical studies [Sittipunt and Wood, 1995] indicated that the hysteretic response of walls susceptible to shear failures could be improved if diagonal reinforcement was used in the web. Diagonal web reinforcement provided a more effective mechanism for transferring lateral forces into the foundation and resulted in lower shear strains near the base of the wall and improved energy dissipation characteristics.

Experience during the 1989 Loma Prieta and 1994 Northridge earthquakes has shown that economic losses can be significant in buildings that satisfied the life safety design criteria inherent to current building codes. As a result, procedures to consider the post-earthquake condition of a building when establishing design limit states are currently being developed. Diagonal web reinforcement in structural walls appears to be one way to control structural damage using conventional methods of construction. The results of an experimental investigation comparing the hysteretic response of walls with conventional and diagonal web reinforcement are described in this paper.

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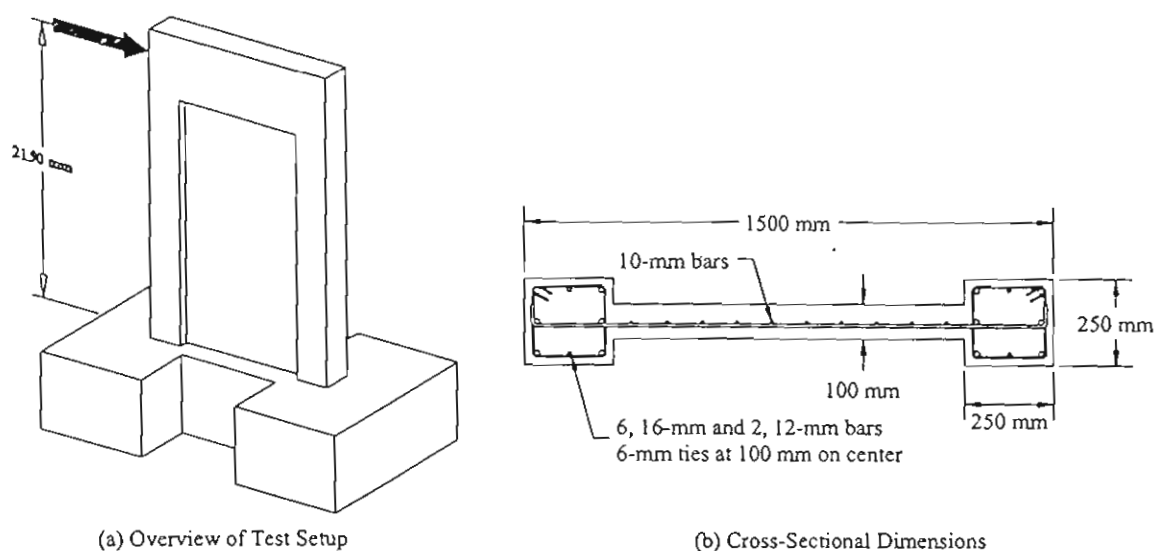


Figure 1: Dimensions of Test Specimens

EXPERIMENTAL PROGRAM

Four reinforced concrete wall specimens were constructed and tested at Chulalongkorn University to investigate the influence of diagonal web reinforcement on the hysteric response of structural walls. The dimensions of the specimens are shown in Figure 1. All walls had a barbell-shaped cross section with a web thickness of 100 mm and 250 by 250-mm boundary elements. The overall length of the cross section was 1500 mm. Vertical reinforcement was anchored in a 600-mm thick base girder that was bolted to the laboratory floor. A 250-mm wide by 500-mm deep beam was cast on top of the wall panel, and a hydraulic actuator was attached to the specimen at mid-depth of the beam. Lateral loads were applied 2150 mm above the base of the wall.

The primary experimental parameters were the amount and orientation of the web reinforcement. The longitudinal and transverse reinforcement in the boundary elements was the same in all four specimens (Fig. 1b). The transverse reinforcement in the boundary elements was not intended to provide confinement of the concrete core. The amount of transverse reinforcement provided was consistent with current practice in Thailand and was approximately one-fifth that required in the U.S. in regions of high seismic risk. A single layer of web reinforcement was used in all walls. Specimens W1 and W2 were reinforced with conventional horizontal and vertical web reinforcement, while diagonal web reinforcement was used in specimens W3 and W4.

The design procedures in the American Concrete Institute Building Code [ACI 318, 1995] for regions of low and moderate seismic risk were used to proportion the walls. During design, the concrete compressive strength was assumed to be 29 MPa and the yield stress of the reinforcement was assumed to be 390 MPa. The measured strengths of the materials used to construct the walls exceeded these values and are summarized in Table 1.

The reinforcement in specimen W1 was selected such that the nominal shear and flexural strengths of the wall were the same. Horizontal web reinforcement was spaced at 150 mm on center, and vertical web reinforcement was spaced at 200 mm on center for specimen W1. Bar spacings were decreased to 100 mm for the horizontal reinforcement and 150 mm for the vertical reinforcement in specimen W2. This change in spacing had a negligible influence on the calculated flexural capacity, but increased the nominal shear strength by 25% relative to specimen W1. The spacing of the web reinforcement in specimens W3 and W4 was the same as the spacing of the horizontal web reinforcement in specimens W1 and W2, respectively (Table 1). However, the web reinforcement in specimens W3 and W4 was rotated 45 degrees with respect to the longitudinal axis of the wall. The nominal strengths of specimens W1 and W3 and specimens W2 and W4 were essentially the same. Nominal capacities calculated using the measured material properties are listed in Table 2.

During the tests, lateral displacements were measured at four locations over the height of the walls (450, 900, 1500, and 2100 mm above the base). In addition, the shear strains were estimated from the diagonal distortion of a 900-mm square located at the base of the wall.

Table 1: Material Properties and Reinforcement Ratios

	W1	W2	W3	W4
Concrete Compressive Strength, MPa	36.6	35.8	37.8	36.3
Horizontal Web Reinforcement				
Spacing, mm	150	100	—	—
Reinforcement ratio, %	0.52	0.79	—	—
Vertical Web Reinforcement				
Spacing, mm	200	150	—	—
Reinforcement ratio, %	0.39	0.52	—	—
Diagonal Web Reinforcement				
Spacing, mm	—	—	150	100
Reinforcement ratio, %	—	—	0.52	0.79
Longitudinal Reinforcement in Boundary Elements [†]				
Area, mm ²	1430	1430	1430	1430
Reinforcement ratio, %	2.29	2.29	2.29	2.29
Transverse Reinforcement in Boundary Elements [‡]				
Spacing, mm	100	100	100	100

Notes:

• Single layer of 10-mm deformed bars, $f_y = 450$ MPa† Six, 16-mm deformed bars, $f_y = 473$ MPa and two, 12-mm deformed bars, $f_y = 425$ MPa‡ Single tie, 6-mm plain bars, $f_y = 444$ MPa

MEASURED RESPONSE

The cyclic loading history used to test the specimens may be divided into discrete stages, each comprising three complete cycles to a specified force or displacement level. During the first stage, the walls were pushed with a maximum force of ± 200 kN, which corresponded to the nominal cracking load. In subsequent stages, the specimens were pushed to integer multiples of the observed yield displacement, beginning with a displacement ductility of 1 in the second stage. The maximum positive displacement resisted by each wall during the first five stages of loading is shown in Figure 2. Slight differences in the maximum displacements sustained by each wall were observed during the first four stages of loading. These variations increased in amplitude during subsequent loading stages. Testing continued until the lateral load capacity of each specimen was reduced by the abrupt failure of the web or boundary elements. Three of the four specimens were able to withstand three complete cycles to a displacement ductility of 4 before failure, while specimen W3 failed during the second cycle to this displacement level.

Continuous plots of load versus top displacement are shown in Figure 3. Significant differences may be observed between the hysteretic response of the walls with conventional web reinforcement (W1 and W2) and the walls with diagonal web reinforcement (W3 and W4). A pinched shape characterized the hysteresis curves for walls W1 and W2. Cracks in the lower portion of the wall did not close when the applied load was reduced to zero, leading to a large reduction in the stiffness of the wall at low levels of applied load. In contrast, the hysteresis curves for walls W3 and W4 exhibited a rounded shape and the effective stiffness did not depend on the magnitude of the applied load during an individual loading cycle. Hysteresis curves for shear distortion in the lower 900 mm of the web are plotted in Figure 4. These data also indicate appreciable improvements in the response due to the diagonal reinforcement.

Table 2: Calculated and Measured Capacity of Specimens

Specimen	Calculated per ACI 318-95		Observed Response		
	Flexural Capacity	Shear Capacity	Maximum Load	Load at Web Crushing	Mode of Failure
	kN	kN	kN	kN	
W1	496	482	491	351	Web Crushing
W2	515	621	608	350	
W3	518	485	569	—	Crushing of Boundary Element
W4	545	622	618	—	

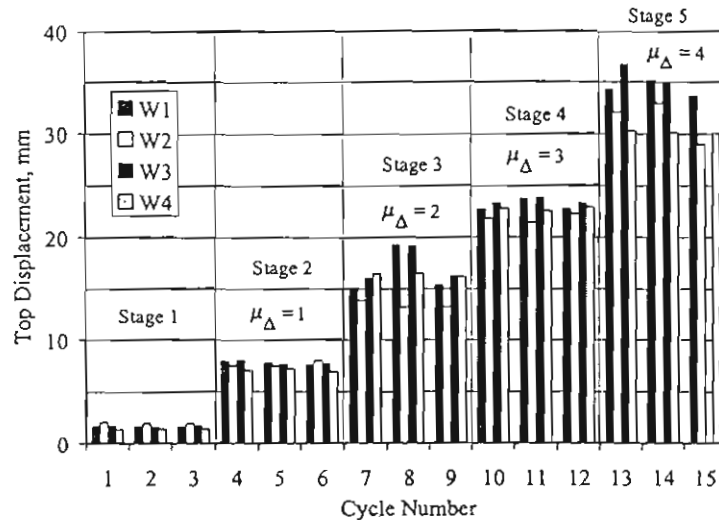


Figure 2: Maximum Positive Displacement Sustained during the First Five Stages of Loading

All four specimens sustained maximum loads that exceeded the calculated nominal capacities (Table 2). The walls with diagonal web reinforcement resisted higher loads than the companion walls with conventional reinforcement; however, the increase in strength was not significant for the walls with higher web reinforcement ratios. Walls W1 and W2 failed abruptly due to web crushing. The applied load at the onset of web crushing was approximately the same for the two specimens. This load was slightly more than 70% of the maximum load resisted by wall W1 and slightly less than 60% of the load resisted by wall W2. Similar reductions in shear strength with cycling have been observed in previous experimental investigations [Wolschlag, 1993]. Walls W3 and W4 failed when the concrete in the boundary elements crushed. This mode of failure was not unexpected, given the modest amount of transverse reinforcement in the boundary elements. Significant increases in the displacement capacities of walls were observed in previous tests when the amount of confinement reinforcement in the boundary elements was increased [Oesterle et al, 1976].

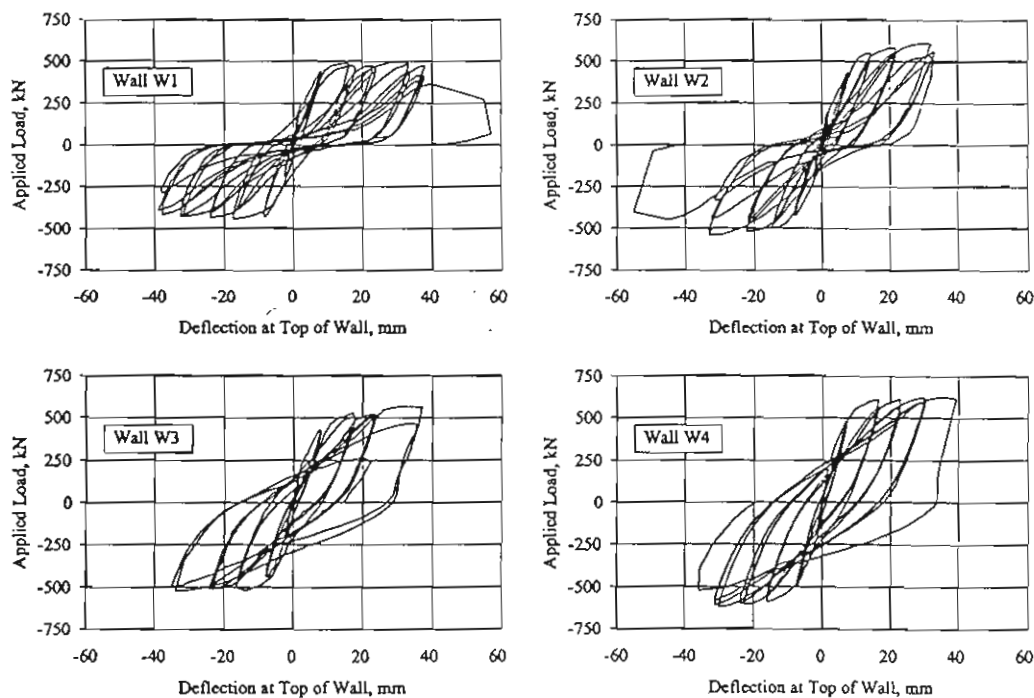


Figure 3: Load vs. Top Displacement

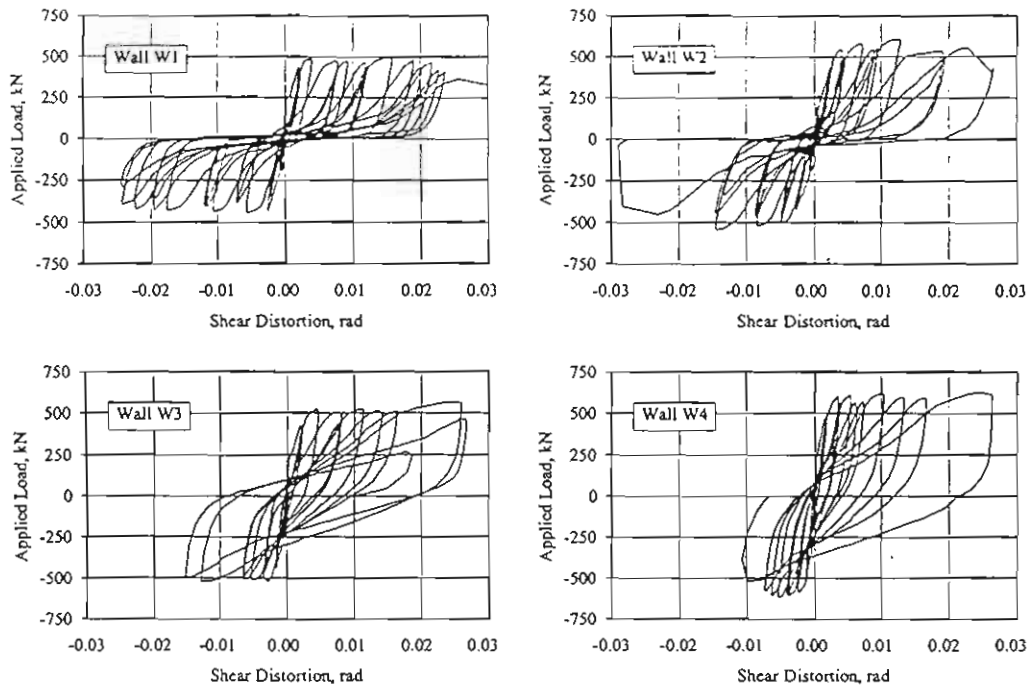


Figure 4: Load vs. Shear Distortion at the Base of the Web

Because the amplitudes of the imposed displacements were not the same for corresponding loading cycles for the four walls, normalized parameters were used to compare the energy dissipation characteristics of the walls. For each loading cycle, the maximum ductility ratio and the area enclosed by the overall hysteresis curves (Figure 3) were calculated. The accumulated ductility ratio was then defined as the maximum ductility ratio for a given cycle plus the sum of the maximum ductility ratios in all previous cycles. Similarly, the accumulated energy was a sum of the area enclosed by the hysteresis loops. These data are plotted in Figure 5 for loading stages 1 through 5. Accumulated energy increased nearly linearly with the accumulated ductility ratio for the four walls. The rate of increase was considerably higher for the walls with diagonal web reinforcement, indicating their ability to dissipate more energy at a given level of distortion. This confirms the qualitative observations based on the shape of the hysteresis curves.

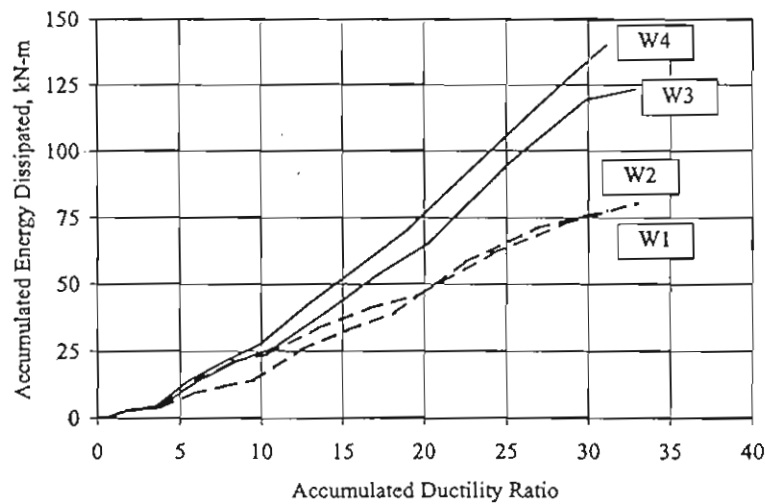


Figure 5: Energy Dissipated during the First Five Loading Cycles

CONCLUSION

The results of this experimental investigation have demonstrated that distributed diagonal web reinforcement is an effective means of improving the hysteretic response of structural walls. Because the diagonal web reinforcement crosses the paths of the cracks nearly perpendicularly, the reinforcement resists the applied loads in tension. In contrast, conventionally reinforced walls must rely on dowel action by the vertical steel and compressive struts in the concrete to transfer the applied loads into the foundation. The strength and stiffness of both of these mechanisms degrade with cycling, and these walls are susceptible to web crushing, a brittle mode of failure. While the capacities of walls W3 and W4 were also limited by the compressive strength of the concrete, previous research has shown that the response can be improved with additional confinement reinforcement in the boundary elements.

As engineers adopt performance-based design philosophies, new techniques must be developed to control structural damage reliably during earthquakes. Diagonal web reinforcement is one such approach. For loading cycles to a specified lateral displacement, walls with diagonal web reinforcement exhibit smaller crack widths and dissipate more energy than conventionally reinforced walls. In addition, with appropriate confinement of the boundary elements, brittle modes of failure can be avoided. These advantages in performance offset the difficulties associated with placement of diagonal bars during construction.

ACKNOWLEDGMENT

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LOAD HISTORY EFFECT ON CYCLIC BEHAVIOR OF R.C. TIED COLUMNS

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Abstract

The effect of two commonly used load histories on the ductility and energy dissipation capacity of reinforced concrete tied columns was investigated. Based on regression analyses of the test results reported in the literature, the effective confinement reinforcement ratio was found to provide a good correlation with the performance indexes studied. The parameters accounted for include the material strength ratio, the amount of the transverse reinforcement, the axial load level and the effectiveness of the tie configurations. Load history with increments of two times the yield displacement was found to be slightly more severe than that with one yield displacement increments.

Introduction

The influence of load history on the cyclic behavior of structures has been recognized for some time. Gosain et al. (1977) proposed the work index as an indicator for evaluating the severity of the loading. They also pointed out limitation of the method when applied to significantly different load histories. Hwang and Scribner (1984) studied the effect of load history on reinforced concrete (r.c.) cantilever beams. The sequence of application of large and small deformations was found to have relatively small effect on the strength and stiffness degradation, and total energy dissipation capacity of the member. The most significant factor was the maximum displacement the members experienced.

The cyclic behavior of reinforced concrete tied columns has been extensively investigated in the literature. In the majority of studies, the material properties, the amount of confinement steel, and the effect of axial load level are normally taken as the main parameters (e.g. Soesianawati, 1986; Azizinamini et al., 1992; Wehbe et al., 1999). The effect of tie configuration effectiveness has recently been considered by Shiekh and Khoury (1997) and Lukkunaprasit et al. (2001). However, no single standard load history has been employed, and it is difficult to compare the performance of r.c. members tested by different investigators. In this paper, the effect of two commonly used load histories on the cyclic inelastic behavior of r.c. columns is reported.

Performance Evaluation

To evaluate the performance of reinforced concrete (r.c.) tied columns on a rational basis, it is necessary to resort to a common and systematic approach in determining the various

response parameters such as the yield displacement, the ultimate displacement, etc. Following the procedures outlined by Sheikh and Khoury (1993) and Legeron and Paultre (2000), the envelope curve (which results from the average of values in both directions) to hysteresis loop is first constructed. In consistence with the method proposed by Watson et al. (1994), the yield displacement, Δ_y , is taken as the displacement value attained at the peak lateral load, H_{max} , assuming a secant stiffness K_y at $0.75 H_{max}$. The displacement ductility factor is defined as

$$\mu_\Delta = \Delta_u / \Delta_y \quad (1)$$

where Δ_u is the displacement reached when the capacity of the specimen dropped to 80% of the peak value which is considered as the failure load.

The dissipated energy is defined as the cumulative energy dissipated within each cycle i . The normalized dissipated energy, E_N , is

$$E_N = (\sum_{i=1}^n E_i) / (H_{max} \Delta_y) \quad (2)$$

where n is the number of cycles to failure and E_i is the dissipated energy in cycle i .

Rather than compute the dissipated energy, Gosain et al. (1977) proposed the work index

$$I_w = \sum_{i=1}^n [(H_{i,max} \Delta_i) / (H_{max} \Delta_y)] \quad (3)$$

as an alternative performance indicator. Eq. (3) was further simplified to

$$N_\Delta = \sum_{i=1}^n (\Delta_i / \Delta_y) \quad (4)$$

which can be interpreted as the cumulative ductility ratio (Sheikh and Khoury, 1993).

Effective confinement steel ratio

Although many researchers consider the effect of material and axial load level on the seismic performance of r.c. tied columns, few explicitly took into account the variation in effectiveness of different tie configurations. Sheikh and Khoury (1997) suggested that the confinement steel as recommended by the ACI Code (1999) be modified by a confinement efficiency parameter, and by nonlinear functions of the axial load level and the curvature ductility demand. Based on the concept of Sheikh and Khoury, Lukkunaprasit et al. (2001) suggested that the effective confinement steel, $A_{sh}(k_e)$, for a targeted ductility demand, μ_Δ , be given by the ACI code value, $A_{sh,ACI}$, modified by the axial load and ductility level functions, i.e.

$$A_{sh}(k_e') = A_{sh,ACI} Y_p g(\mu_\Delta) \quad (5)$$

where k_e' is the confinement effectiveness coefficient normalized with respect to 0.75, which is the upper bound value assumed by the ACI Code for the efficiency of rectilinear ties in comparison with that of closely-spaced spirals; Y_p is the axial load parameter which takes into account the effect of the axial load level (Sheikh and Khourey, 1997) and $g(\mu_\Delta)$ is a nonlinear function of the displacement ductility demand. The simple empirical formula proposed by Saatcioglu (1996) for the confinement effectiveness coefficient, k_e , was used in this study. Eq. (5) can be rewritten as

$$\mu_\Delta = f(A_{sh} k_e' / A_{sh,ACI} Y_p) = f(\rho_A) \quad (6)$$

where $f(\rho_A)$ is a nonlinear function of the effective confinement steel ratio, ρ_A .

Test Samples

Although extensive tests on r.c. tied columns have been conducted, quite a number of test procedures and load histories were followed by various researchers, making comparison of column performance difficult. Based on the number of test results available which needs to be large enough for a regression analysis, and the availability of digitized hysteresis loops, the number of test specimens that could be used in this study is only 16. These specimens can be broadly grouped into two categories in accordance with displacement histories as shown in Fig. 1. History types 1 and 2 pertain to testing with basically two cycles repeated for each displacement increment of $1\Delta_y$ and $2\Delta_y$, respectively. Table 1 lists the details of test specimens whose results are used in the analyses. The main parameters, besides the load histories, include : material strength ratio f_c' / f_{yh} in the range of 0.060-0.136, where f_c' is the concrete compressive strength and f_{yh} is the yield strength of the transverse steel; shear span-to-depth ratio of 3.0 or larger; longitudinal reinforcement ratio of 1.51-3.14%; transverse steel in the range of 29%-176% of that specified in ACI code (1999); axial load level of 0.08-0.70 of the concrete gross section capacity; and tie configurations as shown in Fig. 2. All column specimens were made of normal strength concrete, and all exhibited flexure mode of failure. The data on hysteresis loops were obtained from www.ce.washington.edu/~peera1

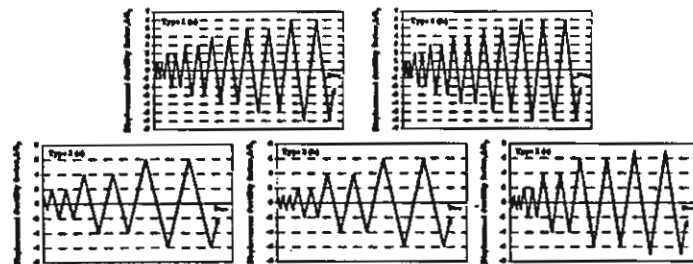


Fig. 1 Displacement histories

Table 1. Details of test specimens used in analyse

Reference	Specimen	A_g (m ²)	Concrete Strength (Mpa)	Longitudinal Steel f_y (Mpa)	ρ (%)	f_{ch} (Mpa)	$A_{ch}/(s \cdot h)$	$A_{ch}/A_{g,act}$	Axial Load Level $P/(f_c' A_g)$	Shear span-to- depth ratio	Tie Config.	Load History Type	k_e
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Socianawati et al. (1986)	UNIT 2	0.160	44.0	444	1.51	360	0.00601	0.55	0.30	4.00	RO	2(a)	0.58
	UNIT 3	0.160	44.0	444	1.51	364	0.00593	0.56	0.30	4.00	RO	2(a)	0.54
Watson et al. (1994)	UNIT 5	0.160	41.0	474	1.51	372	0.00579	0.58	0.50	4.00	RO	1(a)	0.57
	UNIT 6	0.160	40.0	474	1.51	388	0.00575	0.59	0.50	4.00	RO	1(a)	0.52
	UNIT 7	0.160	42.0	474	1.51	308	0.01111	0.91	0.70	4.00	RO	1(a)	0.55
	UNIT 8	0.160	39.0	474	1.51	372	0.00609	0.65	0.70	4.00	RO	1(a)	0.59
	UNIT 9	0.160	40.0	474	1.51	308	0.02051	1.76	0.70	4.00	RO	2(a)	0.71
Azizianini et al. (1992)	NC-3	0.309	39.3	439	1.94	454	0.01152	1.01	0.21	3.00	RD	1(a)	0.43
	NC-4	0.309	39.8	439	1.94	416	0.00650	0.68	0.31	3.00	RD	1(a)	0.42
Wahbe et al. (1999)	A1	0.232	31.7	448	2.22	455	0.00223	0.52	0.08	3.83	RC1	2(b)	0.46
	A2	0.232	27.2	448	2.22	455	0.00223	0.60	0.14	3.83	RC1	1(a)	0.46
	B1	0.232	29.7	448	2.22	455	0.00428	0.77	0.09	3.83	RC1	2(a)	0.53
	B2	0.232	28.1	448	2.22	455	0.00428	0.77	0.23	3.83	RC1	1(b)	0.55
Lukhanaprasit (2001)	CF135/1200.30	0.160	35.7	470	3.14	305	0.00453	0.47	0.30	3.75	RC2	1(a)	0.58
	CFL90/1200.30	0.160	31.7	470	3.14	306	0.00453	0.49	0.30	3.75	RCC	1(a)	0.58
	CFL90/1200.37	0.160	32.4	470	3.14	304	0.00453	0.47	0.37	3.75	RCC	1(a)	0.58

Table 2. Summary of displacement ductility and energy indexes

Reference	Specimen	μ_d	Normalized dissipated energy E_{me}	Cumulative Ductility Ratio N_{me}	Work Index I_{me}
(1)	(2)	(3)	(4)	(5)	(6)
Socianawati et al. (1986)	UNIT 2	5.4	18.2	17.3	15.7
	UNIT 3	5.0	10.1	11.0	10.2
Watson et al. (1994)	UNIT 5	3.4	19.6	19.3	17.7
	UNIT 6	2.6	11.3	12.3	11.2
	UNIT 7	2.8	11.4	8.9	8.3
	UNIT 8	3.1	8.9	8.2	7.6
	UNIT 9	6.3	41.9	27.9	26.0
Azizianini et al. (1992)	NC-3	5.5	52.3	48.9	43.4
	NC-4	3.4	15.0	19.5	17.3
Wahbe et al. (1999)	A1	3.3	11.3	14.2	12.8
	A2	3.4	18.9	21.3	19.6
	B1	3.9	25.4	23.0	20.6
	B2	3.4	29.0	29.9	27.4
Lukhanaprasit (2001)	CF135/1200.30	2.7	9.1	11.6	10.2
	CFL90/1200.30	3.8	18.5	16.1	15.0
	CFL90/1200.37	2.7	9.0	11.1	10.0

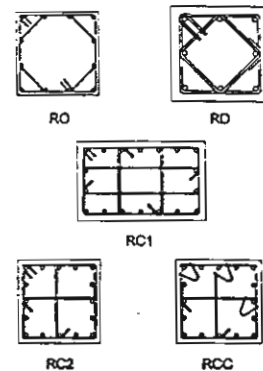
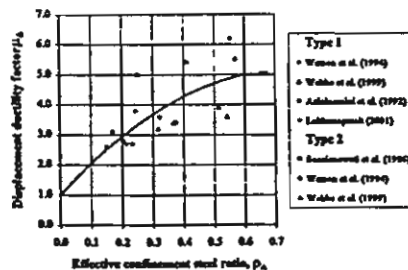


Fig. 2 Tie configurations

Analysis of Results

The calculated values of the μ_d and energy indexes are tabulated in Table 2 and plotted in Fig. 3–Fig. 5 as functions of ρ_A . The plot in Fig. 3 shows that values of μ_d for Type 1 load history are contained in a small band when they are plotted against ρ_A , while those for Type 2 history are much more scattered. A curve fitting of all data points yields

$$\mu_d = 1 + 11.2 \rho_A - 7.59 \rho_A^2 \quad (7)$$

Fig. 3 Displacement ductility factor as function of ρ_A

with the coefficient of determination, r^2 , of 0.44. The rather poor correlation is to be expected because the parameter ρ_A reflects several factors mentioned earlier. Nevertheless, ρ_A is an important parameter which enables comparison of performance of column specimens with different transverse reinforcements (including material properties and tie

arrangements) and different axial load levels.

Clearly more data are definitely needed to reveal the real trend in the results for Type 2 history. For the limited data available, testing with displacement increments of $2\Delta_y$ seems to yield slightly higher ductility factor than testing with $1\Delta_y$ increments.

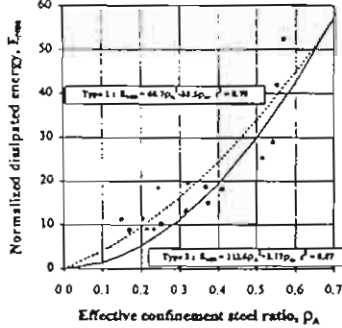


Fig. 4 Normalized dissipated energy as function of ρ_A

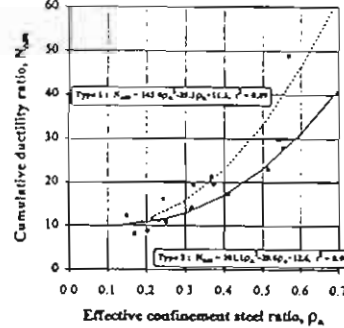


Fig. 5 Cumulative ductility ratio as function of ρ_A

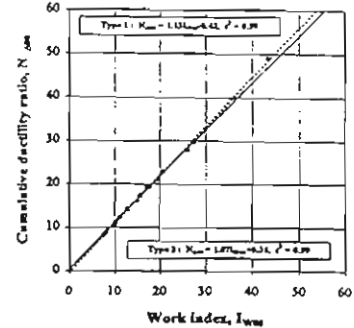


Fig. 6 Correlation between I_{W80} and N_{d80}

The plots in Fig. 4–Fig. 5 show a clear tendency for the normalized energy dissipation capacity and the cumulative ductility ratio to increase nonlinearly with the effective confinement steel ratio. Results of second order curve fitting are shown in the plots.

It is interesting to observe that the load history with increments of $2\Delta_y$ (Type 2) is somewhat more severe than that with $1\Delta_y$ increments (Type 1). For specimens with low effective confinement steel ratio (in the range of 0.15–0.30), the energy dissipation capacity for Type 2 load history is smaller than that for Type 1 load history by about 30%–40%. This discrepancy decreases with increasing effective confinement steel ratio. The difference in cumulative ductility ratio, on the other hand, slowly increases with increasing ρ_A . However, test results at larger values of ρ_A are rather scattered, and it is obvious that more tests are needed on highly confined specimens.

As would be expected, there is a close correlation between the work index and cumulative ductility ratio as depicted in Fig. 6.

Conclusions

The following conclusions can be drawn:

1. For a given load history, the displacement ductility factor, μ_A , and the energy dissipation parameters were found to correlate fairly well with the effective confinement reinforcement ratio. This is of practical significance in enabling prediction of displacement ductility and energy dissipation capacity for a given column, taking into account the effect of axial load and tie configuration effectiveness, among other basic variables.
2. The ductility factor for load history Type 2 (with increments of $2\Delta_y$) tends to be somewhat higher than that for Type 1 (with $1\Delta_y$ increments) for the same effective

confinement steel ratio. However, since the number of samples in the former case is small, further study is needed.

3. The load history with $2\Delta_y$ increments is slightly more severe than that with $1\Delta_y$ increments. The former causes a 30%-40% reduction in the energy dissipation capacity of columns with low effective confinement steel ratio.

Acknowledgments

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ELASTIC SHEAR MODULUS OF CLAYS DURING UNDRAINED SHEARING

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Abstract

The present study aims to monitoring the variation of the elastic shear modulus of clay samples when they were subject to undrained triaxial compression. The elastic shear modulus was measured using a pair of bender elements installed at the top cap and pedestal of the triaxial cell. The isotropic consolidation was conducted step by step so that variation of shear modulus could be determined. It was found that the shear modulus measured during this stage complied well to the information found in the literature. The undrained shearing was carried out at the strain rate of 0.01%/min during which the elastic shear modulus were continuously recorded. It was found that the elastic shear modulus reduced as the pore water pressure increased (or the mean effective stress decreased). The path of the elastic shear modulus according to the mean effective stress repeated that obtained during isotropic consolidation. It implied the negligible effect of the deviator stress. It was further observed that sharp deviation from the isotropic elastic modulus line occurred at high deviator stress level. This can be used to indicate first localization inside the tested sample. However the information obtained from the present arrangement cannot indent soil localized being the location where and properties of soil being localized.

Introduction

It is required the elastic stiffness in the elastic-plastic analysis of a soil undergone shearing. However, most engineers are not paying much attention to the parameter. Comparing to efforts given in formulating the complicated plasticity partner. It is usually treated as a constant for the whole analysis. The elastic stiffness has been known to be functions of several parameters; i.e., effective stress level anisotropy stress condition, Plasticity index, overconsolidation ratio, and etc. Abundant of researches had been done on this and its related topics; e.g., Hardin and Drnevich, 1972, Roesler, 1979, Yu and Richat, 1984, Yan and Byrne, 1991, Jamiolkowski et al. 1991, Tatsuokand shebuya, 1992, etc. Unfortunately, there have been very limit information on variation of elastic stiffness during shearing (Tanizawa et al., 1994, Shibuya and Hwang, 1997, Rampello et al., 1997, Pennington et al., 1997).

This paper aims to address the change of elastic shear modulus of soft Bangkok clay during undergone undrained triaxial compression.

Material and testing program

Undisturbed samples of Bangkok clays were collected from two locations as indicated in Table 1. The basic physical properties were also summarized. Detail information of Bangkok clays is referred to Bergado et al. (1990). A pair of bender elements was installed at the top cap And pedestal of triaxial apparatus. Each element protruded 7 mm into the tested sample at both ends.

Site	Depth (m.)	Natural water content (%)	Plastic Index (%)	Symbol
Tummasartungsit	4.5-5.4	75.54	59.42	□
	10.0-11.4	74.89	60.1	△
	12.0-12.9	31.5	15.48	◇
Chulalongkorn	3.0-4.0	65.65	43.22	○
	6.0-7.0	67.78	58.74	×

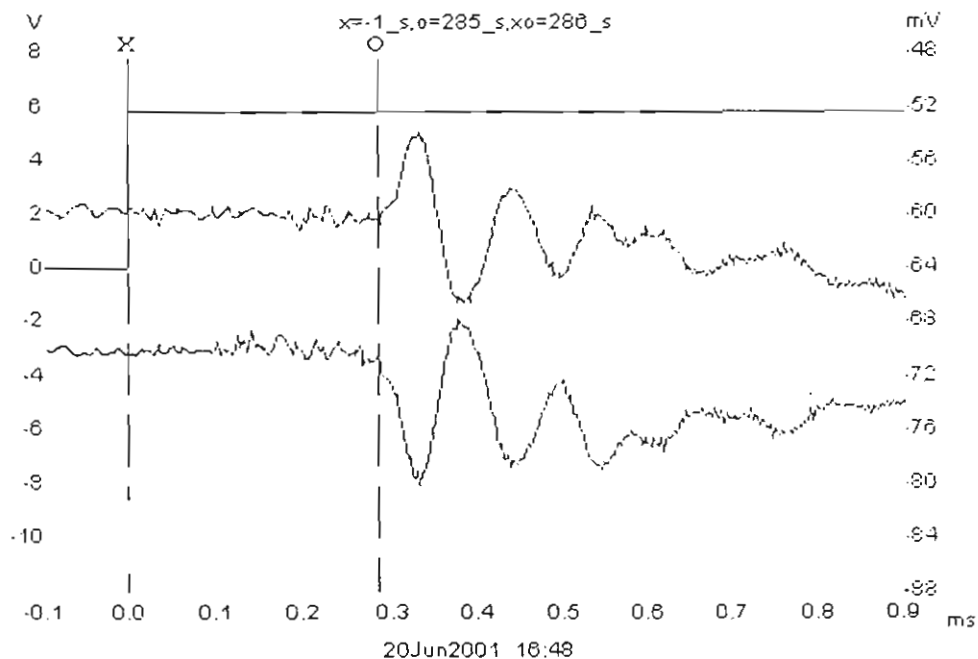


Fig.1 Example of arrival shear wave at the bender element

Test result

The travelling time was determined from the curve such as that shown in Fig.1. The shear modulus can then be computed using;

$$G = \rho v_s^2$$

Fig.2 show the variation of elastic shear moduli against the mean effective stress during isotropic consolidation. The shear modulus has been normalized by a function, $F(e)$, to eliminate the effect of different in void ratio of the sample. The adopted function $F(e)$ is

$$F(e) = \frac{(2.97 - e)^2}{1 + e}$$

There is quite a large scattering of the data points. This is due partly to the effects of different in plasticity and depth of the samples (Table 1). The effect of plasticity seems to be conforming well to that reported in the literature.

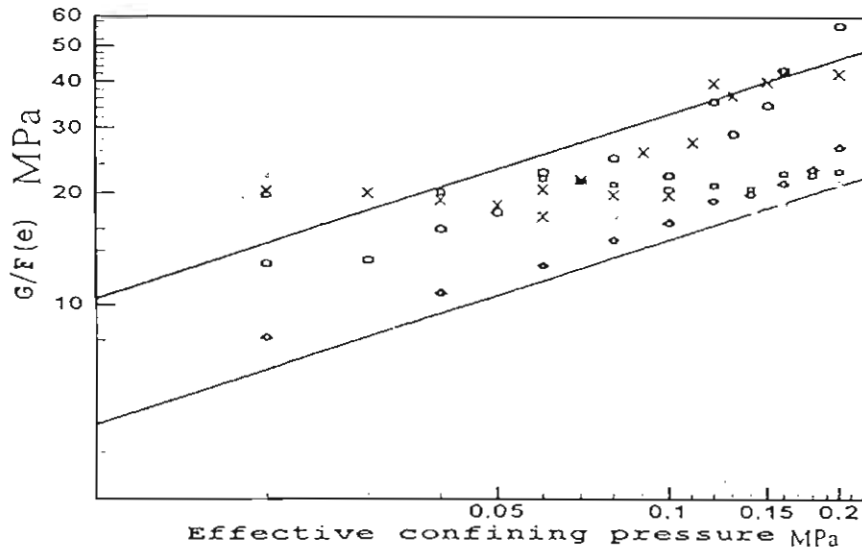


Fig.2 Variation of elastic shear modulus during isotropic consolidation

The variation of elastic shear modulus during the samples undergone undrained compression is addressed in Fig.3 to 5. A line refers to the shear modulus during isotropic consolidation of each sample. It can be seen that the path of shear modulus against the mean effective stress during undrained shearing is almost similar to those obtained during isotropic consolidation. This implies that the effect of the deviator shear stress is very small and can be neglected. It should be noted that when deviator stress achieved its peak value, sudden reduction in the measured shear modulus is observed. This was true for all

samples tested. It is believed that localization (or discontinuity) in the sample is the main reason for such drastic reduction of shear modulus.

Conclusions

The elastic shear modulus obtained during undrained triaxial shearing exhibit similar dependency on the mean effective stress. The path is similar to that obtained during isotropic consolidation. The result implied negligible effect of the deviator stress level. It should be noted the measurement of shear wave propagation velocity can be used to detect the formation of any discontinuity inside the sample,

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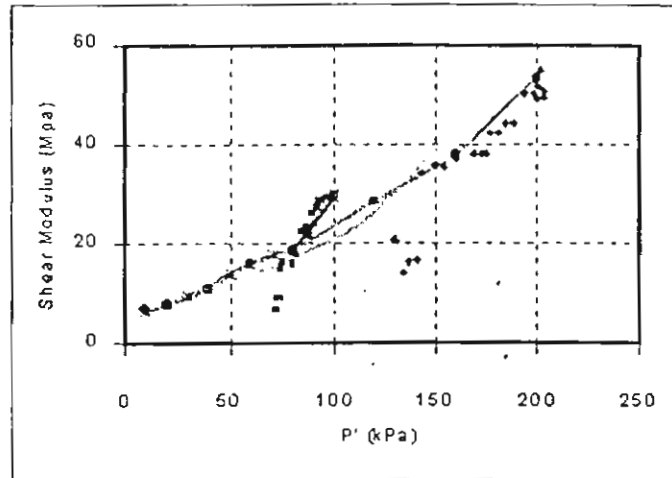


Fig.3 Elastic shear modulus during undrained shearing (CU site -3.0 m)

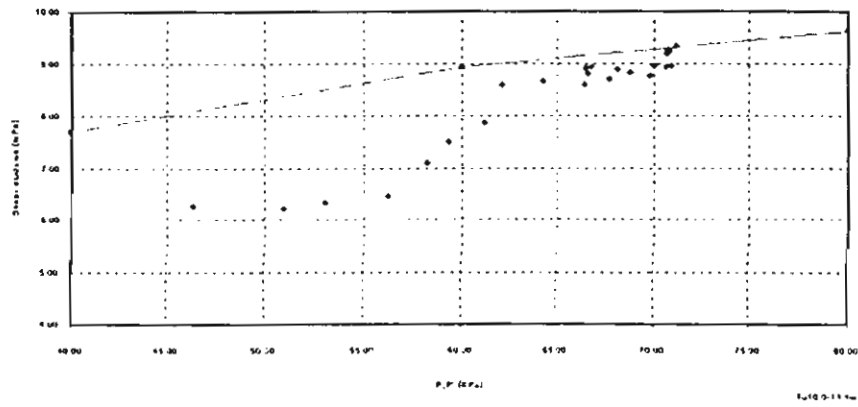


Fig. 4 Elastic shear modulus during undrained shearing (TU site -10 m)

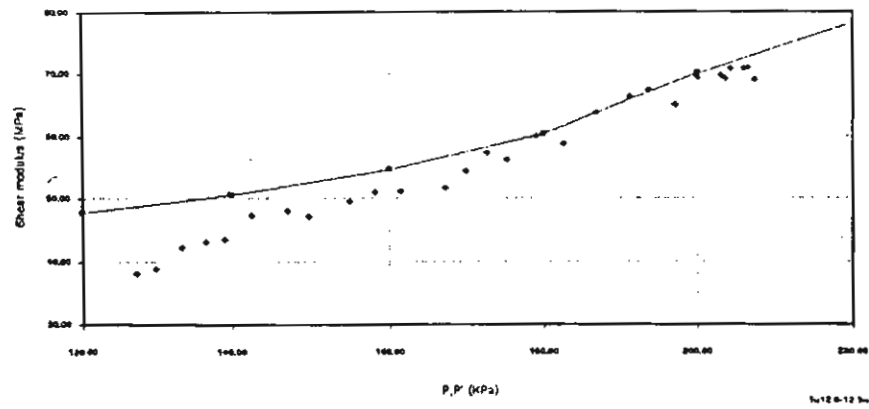


Fig. 5 Elastic shear modulus during undrained shearing (TU site -12 m)

ภาคผนวก จ

บทความที่ตีพิมพ์ในการประชุมวิชาการในประเทศ

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การควบคุมการสั่นไหวของโครงสร้างอีลาสโตพลาสติกภายใต้การเคลื่อนที่ของฐานรองรับ

โดยใช้มวลหน่วงปรับค่า

VIBRATION CONTROL OF ELASTO-PLASTIC STRUCTURES
UNDER SUPPORT MOTION USING TMD

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บทคัดย่อ: บทความนี้ ศึกษาประสิทธิภาพของมวลหน่วงปรับค่าในการลดการสั่นไหวและความเสียหายของโครงสร้างที่มีพฤติกรรมแบบอีลาสโตพลาสติกภายใต้การเคลื่อนที่บริเวณฐานรองรับแบบฮาร์โมนิกและจากสัญญาณแผ่นดินไหวจริง โดยวัตถุประสงค์หลักของการศึกษาก็เพื่อทำความเข้าใจพฤติกรรมและข้อจำกัดของมวลหน่วงปรับค่าในการลดการสั่นไหวของโครงสร้างอีลาสติก

ประสิทธิภาพของ TMD ลดลงเมื่อโครงสร้างมีพฤติกรรมอีลาสติก ทั้งนี้เนื่องจากความไม่เหมาะสมของความต่างเฟสระหว่างการสั่นของโครงสร้างและของ TMD เป็นสำคัญ นอกจากนี้ พลังงานที่สลายไปเนื่องจากการครากของโครงสร้างมีค่ามากกว่าพลังงานที่สลายไปเนื่องจาก TMD ก่อนข้างมาก ดังนั้นพลังงานเนื่องจาก TMD จึงแทบไม่มีนัยสำคัญแต่อย่างใด

อย่างไรก็ตาม ถึงแม้ประสิทธิภาพของ TMD จะค่อยๆลดลงเมื่อแอมพลิจูดของความเร่งที่ฐานเพิ่มขึ้นก็ตาม แต่การติดตั้ง TMD ก็เป็นการเพิ่มความต้านทานให้กับโครงสร้าง

ABSTRACT: This paper studies the effectiveness of TMD in reducing the vibration and the damage of Elasto-Plastic Structure subjected to harmonic and earthquake base excitations.

It is found that the effectiveness of TMD is reduced because of improper phase induced by yielding of the structure. Moreover, the energy caused by yielding of the structure is much larger than the dissipating energy caused by TMD.

Although the effectiveness of TMD is decreased as the support acceleration is increased, the installation of TMD always improves the resistance of structure.

KEYWORDS: Tuned Mass Damper, Elasto-Plastic, Base Excitation

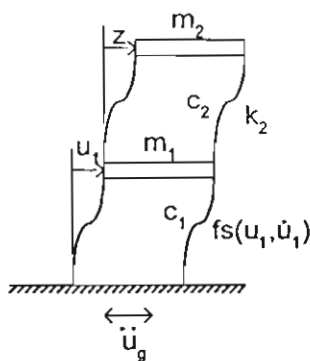
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1. บทนำ

การศึกษาด้านความสามารถของมวลหน่วงปรับค่า (Tuned Mass Damper, TMD) ในการลดการสั่นไหวและเพิ่มความต้านทานให้กับโครงสร้างที่มีพฤติกรรมอินอีลาสติกภายใต้การเคลื่อนตัวที่ฐานเนื่องจากสัญญาณแผ่นดินไหวต่างๆ ในอดีตยังเป็นการศึกษาที่ไม่ลึกซึ้งอีกทั้งผลการศึกษาก็ยังไม่สอดคล้องไปในทางเดียวกันนัก [1,2,3,4] ดังนั้นจึงเป็นสิ่งที่น่าสนใจอย่างยิ่งที่จะทำการศึกษาด้านความสามารถของ TMD ทั้งนี้เพื่อทำความเข้าใจถึงพฤติกรรมของ TMD และเป็นพื้นฐานในการพัฒนาประสิทธิภาพของ TMD ต่อไป

2. การวิเคราะห์การสั่นไหวของโครงสร้างอีลาสโตพลาสติกที่ติดตั้งระบบ TMD

2.1 สมการการเคลื่อนที่



จากรูปที่ 1 สามารถเขียนสมการสมดุลของโครงสร้างและของ TMD ได้ดังสมการ 1 และ สมการ 2 ตามลำดับ

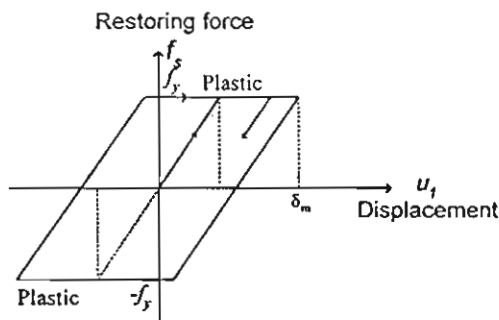
$$m_1 \ddot{u}_1 + c_1 \dot{u}_1 + f_s(u_1, \dot{u}_1) = -m_1 \ddot{u}_g + c_2 \dot{z} + k_2 z \quad (1)$$

$$m_2 \ddot{z} + c_2 \dot{z} + k_2 z = -m_2 \ddot{u}_1 - m_2 \ddot{u}_g \quad (2)$$

รูปที่ 1 ระบบโครงสร้างที่ติดตั้ง

โดย $f_s(u_1, \dot{u}_1)$ คือ แรงต้านทานภายในเนื่องจากสตีเฟนสของโครงสร้าง

2.2 ระบบอีลาสโตพลาสติก (Elasto-Plastic System)



รูปที่ 2 แบบจำลองความสัมพันธ์ระหว่าง f_s และ u_1 ในระบบอีลาสโตพลาสติก

คือระบบที่ $f_s(u_1, \dot{u}_1)$ เป็นแบบอินอีลาสติกอย่างง่าย โดยสามารถแบ่งได้เป็น 2 ส่วน คือ ส่วนที่มีสภาพอีลาสติกและส่วนที่มีสภาพอินอีลาสติก โดยเมื่อโครงสร้างมีการกระจัดน้อยกว่าการกระจัดที่จุดคราก โครงสร้างจะมีสภาพอีลาสติก และหากการเคลื่อนที่ถึงจุดที่การกระจัดที่จุดคราก โครงสร้างจะมีพฤติกรรมเปลี่ยนเป็นอินอีลาสติกโดยแรงต้านทานเนื่องจากสตีเฟนสของโครงสร้างจะมีค่าคงที่ จนกระทั่งโครงสร้างไปถึงตำแหน่งการกระจัดสูงสุดในรอบนั้นความเร็วของการ

เคลื่อนที่เปลี่ยนทิศทางโครงสร้างจะกลับมามีสภาพอีลาสติกอีกครั้งหนึ่ง ซึ่งจะมีพฤติกรรมแบบนี้ไปเรื่อยๆ โดยพลังงานที่สลายออกมาเนื่องจากการครากของโครงสร้าง (Hysteresis Energy) จะมีค่าเท่ากับพื้นที่ใต้กราฟความสัมพันธ์ของแรงภายในและการกระจัดในรอบนั้นๆ (Hysteresis Loop) ดังแสดงในรูปที่ 2 ซึ่งจะบ่งชี้ถึงปริมาณความเสียหายที่เกิดขึ้นในโครงสร้างด้วย

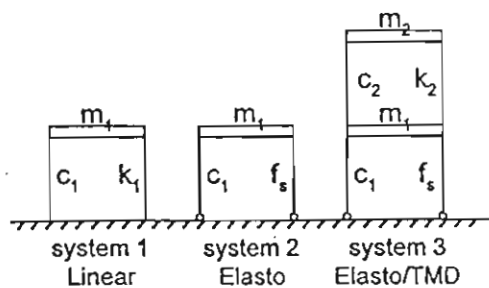
2.3 คำนวณความเสียหาย (Damage Index)

เป็นปริมาณที่บ่งชี้ถึงความเสียหายของโครงสร้าง โดยพิจารณาจากการกระจัดสูงสุดและค่าพลังงานที่สลายออกมาเนื่องจากการครากของโครงสร้าง ที่นำเสนอโดย Park and Ang [5] ดังสมการ 3

$$DI = \frac{\delta_m}{\delta_u} + \beta_c \frac{\int dE}{f_y \cdot \delta_u} \quad (3)$$

โดยที่ δ_m คือ การกระจัดสูงสุดของการเคลื่อนที่, δ_u คือ การกระจัดประลัย, β_c มีค่าเฉลี่ยเท่ากับ 0.27, $\int dE$ คือ พลังงานที่สลายออกมาเนื่องจากการเกิดคราก, f_y คือ แรงต้านทานที่จุดครากของโครงสร้าง

2. แบบจำลองโครงสร้างและ TMD



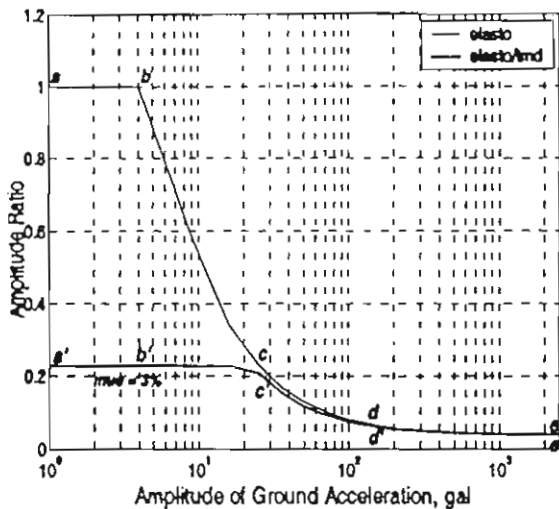
รูปที่ 3 แบบจำลองโครงสร้างทั้ง 3 ระบบ

ในการศึกษานี้จะพิจารณาแบบจำลองโครงสร้าง 3 ระบบดังแสดงในรูปที่ 3 โดยมีมวล $m_1 = 471.03$ ตัน ความถี่เชิงมุม 1.78 เฮิรตซ์ อัตราส่วนความหน่วง 2 เปอร์เซ็นต์ ค่าสทิฟเนสในช่วงอีลาสติก 1492.41 นิวตัน/เมตร และแรงต้านทานของโครงสร้างในช่วงอินอีลาสติกมีค่า 60,000 นิวตัน โดยมีความถี่ของการเคลื่อนที่ฐานของโครงสร้างเป็นฮาร์โมนิกที่มีความถี่ในการสั่นเท่ากับความถี่เชิงมุมของแบบจำลอง เพื่อให้เกิดการสั่นพ้องขึ้น (Resonance) ส่วนค่าพารามิเตอร์ต่างๆของ TMD สามารถคำนวณได้จากสมการของ Den's Hartog [6] โดยเลือกใช้ค่าอัตราส่วนของมวลเท่ากับ 3 เปอร์เซ็นต์และให้ TMD มีพฤติกรรมอยู่ในช่วงอีลาสติก

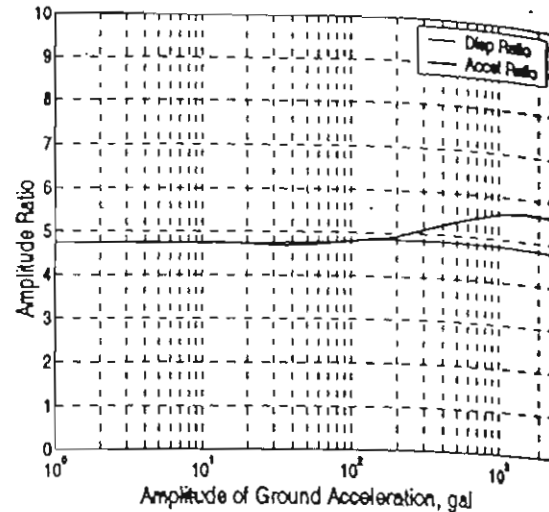
3. ผลการศึกษา

3.1 การศึกษาประสิทธิภาพในการควบคุมการสั่นไหวของโครงสร้างภายใต้การสั่นที่ฐานแบบฮาร์โมนิก

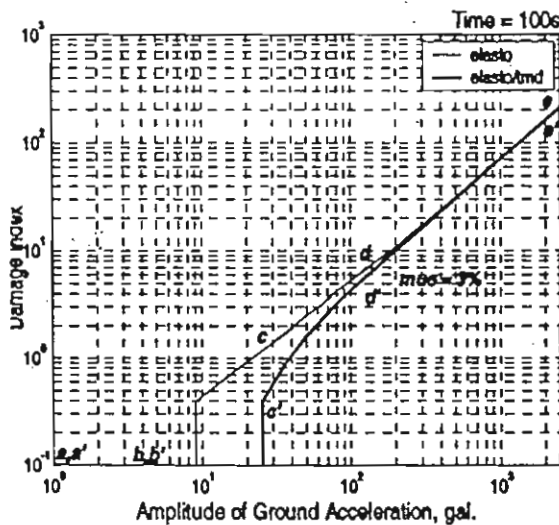
ค่าอัตราส่วนระหว่างแอมพลิจูดของการกระจัดที่สภาวะคงตัวของโครงสร้างระบบที่ 2 ต่อระบบที่ 1 และระบบที่ 3 ต่อระบบที่ 1 ที่แอมพลิจูดของความถี่ฐานของโครงสร้างค่าต่างๆแสดงในรูปที่ 4(a) คิวเส้น a-b-c-d-e และเส้น a'-b'-c'-d'-e' ตามลำดับ และความสัมพันธ์ระหว่างดัชนีความเสียหายของโครงสร้างที่เวลา 100 วินาทีของโครงสร้างระบบที่ 2 และระบบที่ 3 ที่แอมพลิจูดของความถี่ฐานของโครงสร้างค่าต่างๆแสดงในรูปที่ 4(b) พบว่าที่แอมพลิจูดของความถี่ฐานค่าน้อยๆ (a-b) โครงสร้างระบบที่ 2 สั่นอยู่ในช่วงอีลาสติก ค่าดัชนีความเสียหายมีค่าเท่ากับศูนย์ ดังนั้นเมื่อติดตั้ง TMD ที่เหมาะสมเข้าไป จะสามารถลดการสั่นไหวของโครงสร้างได้อย่างมีนัยสำคัญ เมื่อเพิ่มขนาดของแอมพลิจูดของความถี่ฐานสูงขึ้น (b-c) โครงสร้างระบบที่ 2 มีการสั่นไหวเกินจุดคราก ทำให้โครงสร้างเกิดความเสียหาย ดังจะเห็นได้จากค่าดัชนีความเสียหายที่เพิ่มขึ้น เมื่อเกิดการแตกร้าวจะมีการสลายพลังงานออกไป ทำให้อัตราส่วนของแอมพลิจูดมีค่าน้อยกว่า 1 ในขณะที่โครงสร้างระบบที่ 3 ยังคงสั่นไหวในช่วงอีลาสติก ไม่เกิดความเสียหายใดๆ



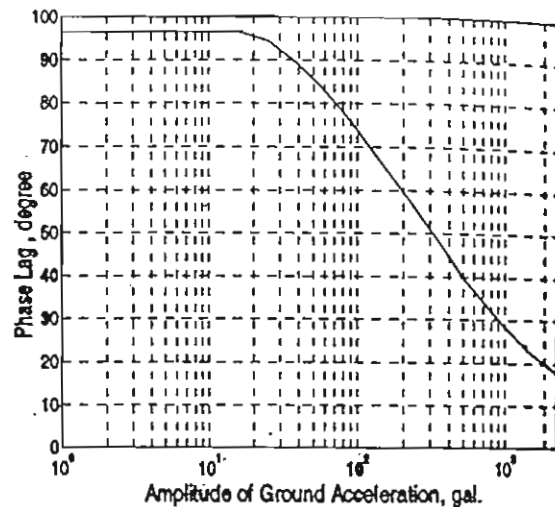
(a)



(c)



(b)



(d)

รูปที่ 4(a) ค่าอัตราส่วนแอมพลิจูดของระบบที่ 2 และระบบที่ 3 ต่อระบบที่ 1 ที่ความเร่งที่ฐานของโครงสร้างค่าต่างๆ, 4(b) ความสัมพันธ์ระหว่างค่าดัชนีความเสียหายของโครงสร้างระบบที่ 2 และระบบที่ 3 และแอมพลิจูดของความเร่งที่ฐานค่าต่างๆ, 4(c) ความสัมพันธ์ระหว่างอัตราส่วนการกระจัดของ TMD ต่อการกระจัดที่ฐานของ TMD, 4(d) ความต่างเฟสระหว่างการสั่นไหวของโครงสร้างและ TMD ที่แอมพลิจูดของความเร่งที่ฐานค่าต่างๆ

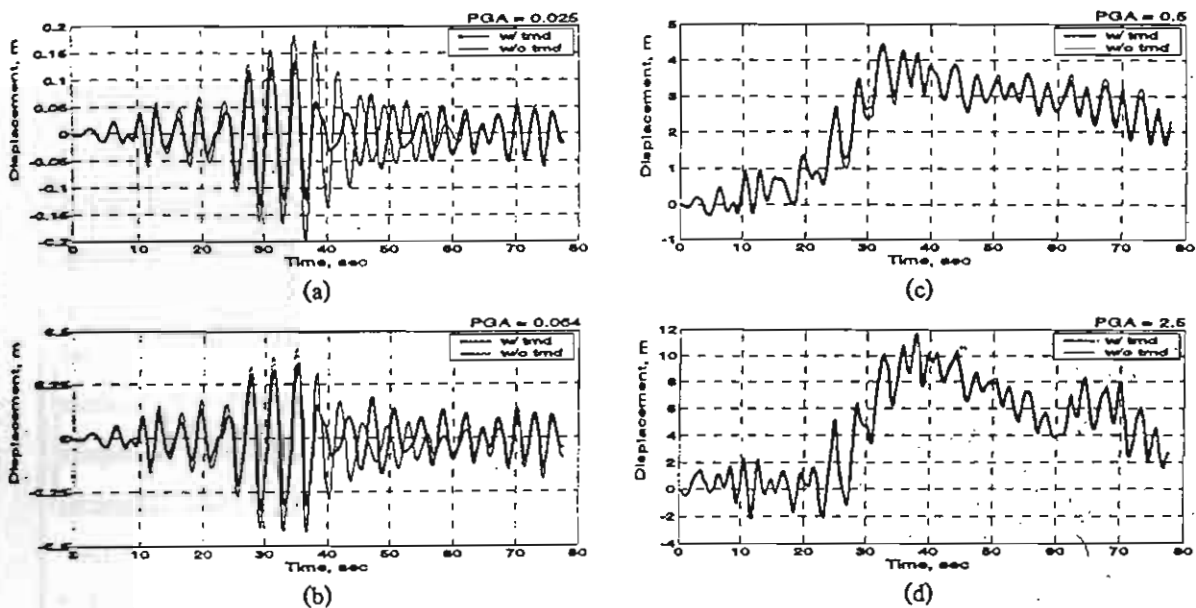
เมื่อขนาดแอมพลิจูดของความเร่งที่ฐานเพิ่มขึ้นอีก (c-d) โครงสร้างระบบที่ 2 และ 3 มีการสั่นไหวเกินจุดคราก ขนาดของแอมพลิจูดของระบบที่ 3 น้อยกว่าระบบที่ 2 เล็กน้อย นอกจากนี้ ค่าดัชนีความเสียหายของระบบที่ 3 ยังคงน้อยกว่าของระบบที่ 2 เล็กน้อยเช่นกัน

อย่างไรก็ตาม เมื่อแอมพลิจูดของความเร่งที่ฐานสูงมาก (d-c) อัตราส่วนแอมพลิจูดและค่าดัชนีความเสียหายของระบบที่ 3 มีค่าใกล้เคียงกับระบบที่ 2 ซึ่งชี้ให้เห็นว่า เมื่อโครงสร้างมีสภาพอินelasติกมากขึ้น ผลของ TMD ในการลดการสั่นไหวและค่าดัชนีความเสียหายให้กับโครงสร้างจะลดลงเรื่อยๆ

จากผลข้างต้น จึงทำการศึกษาถึงการทำงานของ TMD โดยพิจารณาการกระจัดและความต่างระหว่างเฟสการสั่นของโครงสร้างและเฟสการสั่นของ TMD ดังแสดงในรูปที่ 4(c) และ 4(d) ตามลำดับ ซึ่งพบว่าอัตราส่วนของการกระจัดของ TMD ต่อการกระจัดที่ฐานของ TMD ยังคงมีค่าใกล้เคียงกับค่าในช่วงอิลาสติก ซึ่งแสดงว่า TMD ยังคงมีระยะการกระจัดที่ค้ำอยู่ แต่เมื่อพิจารณาความต่างเฟสของการสั่น พบว่าความต่างเฟสจะมีค่าลดลงเมื่อโครงสร้างมีสภาพอิลาสติกเพิ่มขึ้น ทำให้ประสิทธิภาพในการลดการสั่นไหวของ TMD ลดลง เพราะแรงเนื่องจาก TMD ที่กระทำต่อโครงสร้างเป็นจังหวะที่ไม่เหมาะสม

3.2 การศึกษาประสิทธิภาพในการควบคุมการสั่นไหวของโครงสร้างภายใต้แรงแผ่นดินไหว

เพื่อศึกษาถึงประสิทธิภาพในการลดการสั่นไหวของ TMD ภายใต้แรงแผ่นดินไหว จึงได้ใช้แบบจำลองข้างต้นศึกษาโดยใช้สัญญาณคลื่นแผ่นดินไหวที่วัดได้ในกรุงเทพมหานคร ซึ่งการสั่นไหวของโครงสร้างที่ไม่ได้ควบคุมและถูกควบคุมที่แอมพลิจูดของความเร่งที่ฐานค่าต่างๆแสดงในรูปที่ 5 และรูปที่ 6(a) และ 6(b) แสดงถึงค่าอัตราส่วนของการกระจัดสูงสุดของโครงสร้างและค่าดัชนีความเสียหายที่ความเร่งที่ฐานค่าต่างๆ ตามลำดับ



รูปที่ 5 การสั่นไหวของโครงสร้างเมื่อติดตั้งและไม่ติดตั้ง TMD ที่แอมพลิจูดของความเร่งที่ฐานค่าต่างๆ

จากกราฟในรูปที่ 6 ในช่วงอิลาสติก TMD สามารถลดการกระจัดสูงสุดได้อย่างมีนัยสำคัญ และเมื่อสัญญาณแผ่นดินไหวมีความรุนแรงมากขึ้น ประสิทธิภาพในการลดการสั่นไหวและค่าดัชนีความเสียหายของโครงสร้างจะมีค่าลดลงเรื่อยๆ ทั้งนี้สามารถอธิบายได้ในทำนองเดียวกันกับกรณีที่มีความเร่งที่ฐานเป็นแบบฮาร์โมนิก นอกเหนือจากนี้ ถึงแม้ว่าความรุนแรงของการสั่นบางค่าที่อัตราส่วนการกระจัดสูงสุดของระบบที่ 3 มีค่าสูงกว่าระบบที่ 2 ก็ตาม แต่ค่าดัชนีความเสียหายของโครงสร้างของระบบที่ 3 ก็ยังคงมีค่าน้อยกว่าระบบที่ 2 ซึ่งเป็นการยืนยันได้ถึงข้อดีของการติดตั้ง TMD ให้กับโครงสร้าง

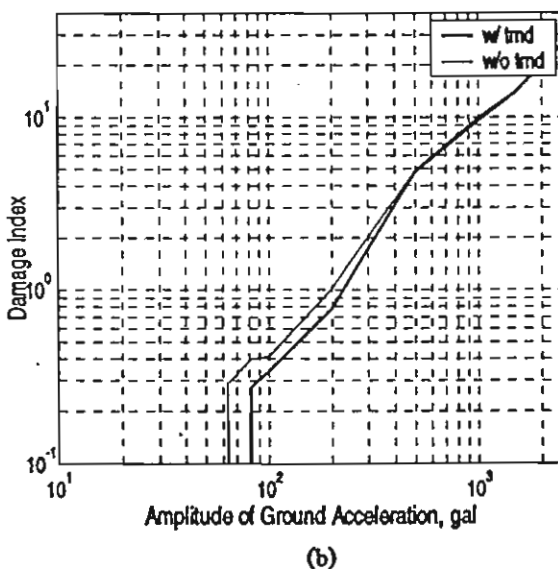
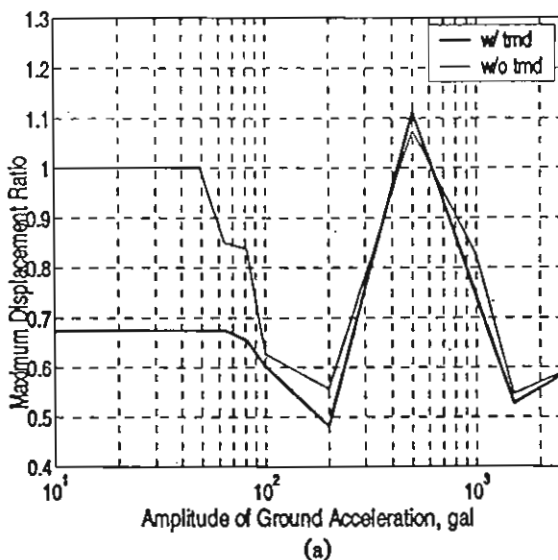
4. สรุป

ประสิทธิภาพในการลดการสั่นไหวของ TMD สำหรับโครงสร้างที่มีพฤติกรรมอินอีลาสติกที่มีการเคลื่อนที่ที่ฐานของโครงสร้างจะมีค่าลดลงเรื่อยๆเมื่อขนาดความรุนแรงของการสั่นที่ฐานมีค่าเพิ่มขึ้น ทั้งนี้เป็นผลเนื่องจากความต่างเฟสระหว่างการสั่นของโครงสร้างและการสั่นของ TMD จะมีจังหวะที่ไม่สอดคล้องกันมากขึ้นเรื่อยๆ นอกจากนี้ พลังงานที่สลายออกไปเนื่องจากการครากของโครงสร้างมีค่าค่อนข้างมากเมื่อเทียบกับพลังงานที่สลายออกไปโดย TMD จึงทำให้ TMD ลดการสั่นไหวของโครงสร้างได้น้อยลง

แต่อย่างไรก็ตาม ถึงแม้การสั่นไหวจะลดลงเพียงเล็กน้อย แต่ TMD ก็สามารถลดค่าความเสียหายที่เกิดขึ้นกับโครงสร้างได้พอสมควร ซึ่งนับเป็นข้อดีของ TMD ต่อโครงสร้าง

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รูปที่ 6 อัตราส่วนการกระจัดสูงสุดและค่าดัชนีความเสียหายของโครงสร้างภายใต้สัญญาณแผ่นดินไหวที่ กทม.

การออกแบบมวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟเพื่อการควบคุมการสั่นไหวของอาคาร DESIGN OF SEMI-ACTIVE TUNED MASS DAMPER FOR VIBRATION CONTROL OF BUILDINGS

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จุฬาลงกรณ์มหาวิทยาลัย กรุงเทพมหานคร

บทคัดย่อ: บทความนี้ได้เสนอการออกแบบมวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟเพื่อควบคุมการสั่นไหวของอาคาร โดยการพิจารณาผลของค่าพารามิเตอร์ที่ใช้ตามวิธีการควบคุมแบบ Linear Quadratic Optimal Control และช่วงการปรับค่าความหน่วงของตัวหน่วงที่มีต่อประสิทธิภาพของมวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟ เพื่อให้สามารถใช่มวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟให้มีประสิทธิภาพสูงสุด นอกจากนี้ยังได้เปรียบเทียบประสิทธิภาพในการลดการสั่นไหวของมวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟที่ได้ออกแบบกับมวลหน่วงปรับค่าแบบพาสซีฟหนึ่งหน่วยและหลายหน่วย ซึ่งพบว่ามวลหน่วงปรับค่าแบบกึ่งแอ็กทีฟสามารถลดการสั่นไหวของอาคารได้มากกว่า

ABSTRACT: This paper proposes the design of semi-active tuned mass damper (STMD) for vibration control of buildings. To achieve the highest effectiveness, the involved parameters in Linear Quadratic Optimal Control and a range of adjustable damping are considered. The vibration controls using STMD, tuned mass damper (TMD) and multi-tuned mass damper (MTMD) are compared. The results reveal that STMD is the most effective.

KEYWORDS :

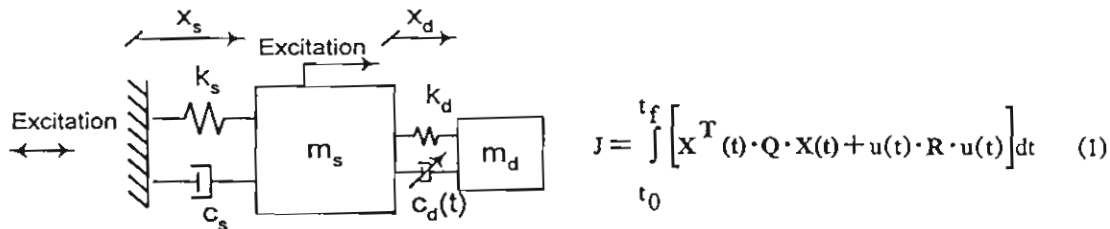
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1. บทนำ

การควบคุมการสั่นไหวของโครงสร้างโดยใช้มวลห่น่วงปรับค่าแบบแอสลึฟ(TMD)และแบบแอคทีฟ(ATMD)ได้มีการศึกษาค้นคว้าและนำไปประยุกต์ใช้ในทางปฏิบัติซึ่งมีผลทำให้สามารถควบคุมการสั่นไหวของอาคารภายใต้แรงลมได้ผลดี อย่างไรก็ตามในทางปฏิบัติพบว่าทั้งการใช้ TMD และ ATMD มีจุดด้อยบางประการ โดยที่ระบบ TMD มีประสิทธิภาพด้อยกว่าส่วนระบบ ATMD ถึงแม้จะมีประสิทธิภาพสูงแต่ในทางปฏิบัติพบว่าต้องเสียค่าใช้จ่ายมากและมีความไม่ปลอดภัย ดังนั้นการประยุกต์ใช้ STMD จึงเป็นวิธีที่น่าสนใจและมีความเป็นไปได้ในทางปฏิบัติเนื่องจากว่ามีประสิทธิภาพสูงในการควบคุมการสั่นไหวของโครงสร้างทั้งยังใช้ค่าใช้จ่ายในการทำงานต่ำและมีความปลอดภัย [3] อย่างไรก็ตามประสิทธิภาพของ STMD นั้นขึ้นอยู่กับคุณสมบัติเชิงกลและพารามิเตอร์ที่ใช้ในการออกแบบระบบควบคุมเป็นอย่างมาก ประกอบกับในอดีตไม่มีการศึกษาถึงวิธีการออกแบบ STMD ดังนั้นการศึกษาดังปจจัยต่างๆที่เกี่ยวข้องกับการออกแบบระบบมวลห่น่วงปรับค่าแบบกึ่งแอคทีฟก็จะสามารถทำให้ใช้ระบบนี้ได้อย่างมีประสิทธิภาพสูงสุด

2. ระบบมวลห่น่วงปรับค่าแบบกึ่งแอคทีฟ (STMD)

การควบคุมการสั่นไหวในระบบ STMD มีวัตถุประสงค์หลักเพื่อลดการสั่นไหวของโครงสร้างหลัก(m_s) สามารถทำได้โดยใช้ algorithm ตามหลัก Linear optimal control ซึ่งเป็นวิธีที่แพร่หลายในการควบคุมแบบแอคทีฟ [4] สามารถหาแรงควบคุม, $u(t)$, ได้จาก Control algorithm ที่ทำให้ performance index, J , มีค่าต่ำที่สุด โดยเลือก J เป็นพลังงานที่ถ่ายเทเข้าสู่โครงสร้างนับตั้งแต่เวลาเริ่มต้น (t_0) จนถึงเวลาสุดท้ายที่สนใจ (t_f) ดังสมการที่ 1



รูปที่ 1 แสดงภาพระบบ STMD

$X(t) = [x_s \ x_d \ \dot{x}_s \ \dot{x}_d]^T$ เป็น response ของระบบซึ่งประกอบไปด้วย displacement และความเร็วของโครงสร้างและมวลห่น่วง $u(t)$ เป็นแรงที่ต้องการในการลดการสั่นของโครงสร้าง Q และ R เป็น weight matrix สำหรับ response ของโครงสร้างและพลังงานที่ใช้ในการควบคุมการสั่นไหว โดยอาศัยการควบคุมด้วยหลักการ feedback สามารถเขียนแรงควบคุมได้ในเทอมของ response ของระบบที่เวลาใดๆคือ

$$u(t) = G \cdot X(t) \quad (2)$$

ค่าเมตริก G ในสมการที่ 2 เป็น gain matrix ซึ่งสามารถหาค่าตอบได้ตามหลัก Optimization จากการเลือกเมตริก Q และ R ค่าหนึ่งๆ ซึ่งเมื่อนำไปคูณกับ response ของระบบตามสมการที่ 2 แล้วจะได้ค่าแรงควบคุมที่เหมาะสม แต่เนื่องจากการให้แรงควบคุมของ STMD นั้นเกิดจากการปรับเปลี่ยนค่าความห่น่วง

$$u(t) = c_d(t) \cdot \dot{x}_d(t) \quad (3)$$

จากสมการที่ 2 และ 3 จะได้ว่าค่าความห่น่วงที่เหมาะสมมีค่าเป็น

$$c_d(t) = \frac{G \cdot X(t)}{\dot{x}_d} \quad ; \quad \dot{x}_d \neq 0 \quad (4)$$

อย่างไรก็ดีในทางปฏิบัติการปรับค่าความหน่วงนั้นมิชอบเขตจำกัด จึงกำหนดให้ค่าความหน่วงที่เหมาะสมเป็นคังสมการ

$$c_{d,min} < c_d(t) = \frac{u(t)}{\dot{x}_d(t)} < c_{d,max} \quad (5)$$

โดยที่ $c_{d,min}$ และ $c_{d,max}$ คือ ค่าสัมประสิทธิ์ความหน่วงต่ำที่สุดและสูงที่สุดที่ระบบ STMD ปรับได้

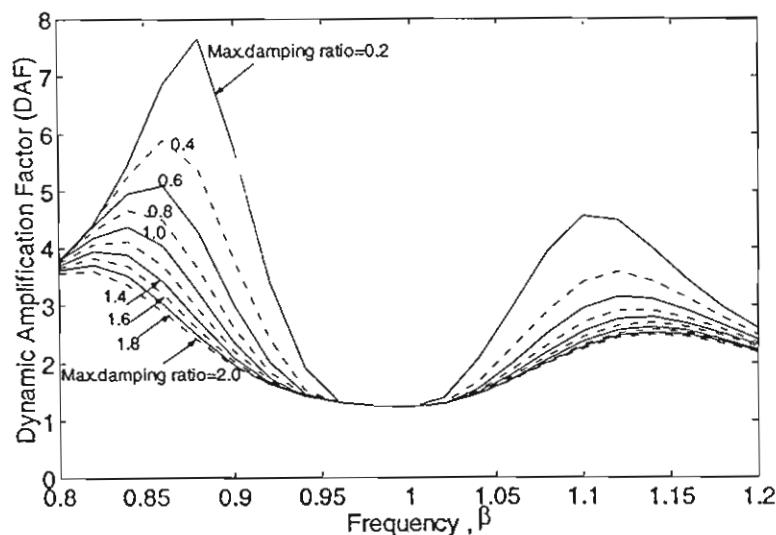
3. ปัจจัยที่มีผลต่อประสิทธิภาพของ STMD

จากการศึกษาพบว่าปัจจัยที่มีอิทธิพลต่อประสิทธิภาพของ STMD คือการเลือกค่า R และช่วงความสามารถในการปรับค่าความหน่วงของ STMD ในบทความนี้ได้แสดงผลการศึกษาอิทธิพลของพารามิเตอร์เหล่านี้ในเชิงตัวเลข โดยจำลองโครงสร้างเป็นระบบเชิงเส้น ค่าพารามิเตอร์ของโครงสร้างและมวลหน่วงแสดงในตารางที่ 1

ตารางที่ 1 แสดงค่าพารามิเตอร์ต่างๆที่ใช้ในการคำนวณ

โครงสร้างหลัก :	$m_s = 471.03 \text{ tons}$	$k_s = 3934.8 \text{ kN/m}$	$c_s = 54.46 \text{ kN/(m/s)}$
STMD :	$m_d = 14.10 \text{ tons}$	$k_d = 111.2 \text{ kN/m}$	
	$c_{d,min} \leq c_d(t) \leq c_{d,max}$		
	$Q = \begin{bmatrix} k_s & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & m_s & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix}$		

3.1 อิทธิพลของช่วงการปรับค่าความหน่วงต่อประสิทธิภาพของระบบ STMD

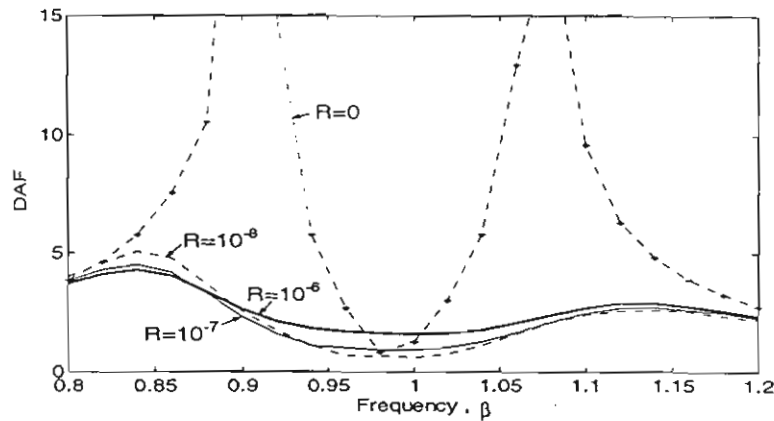


รูปที่ 2 แสดง Frequency-response Curveของการเลือกความสามารถในการปรับค่าความหน่วงของ STMD

รูปที่ 2 แสดงค่า Dynamic Amplification factor (DAF) ของการสั่นไหวกับอัตราส่วนของความถี่ของแรงที่มากระทำต่อความถี่โครงสร้าง(β) จากผลที่ได้แสดงให้เห็นว่าเมื่อระบบ STMD มีความสามารถในการปรับค่าความหน่วงมากขึ้นก็จะมีผลทำให้ประสิทธิภาพของระบบ STMD ค่อนข้างดี โดยเฉพาะในช่วงต้นที่ความสามารถในการปรับความหน่วงมีค่าน้อย เมื่อเพิ่มความสามารถในการปรับนี้ให้มากขึ้นจะพบว่าระบบ STMD มีประสิทธิภาพดีขึ้นมาก แต่ในช่วงที่ระบบ STMD มีความสามารถในการปรับความหน่วงสูงอยู่แล้วนั้นการปรับเพิ่มความสามารถในการปรับความหน่วงให้มากขึ้นทำให้ประสิทธิภาพดีขึ้นเพียงเล็กน้อยเท่านั้น

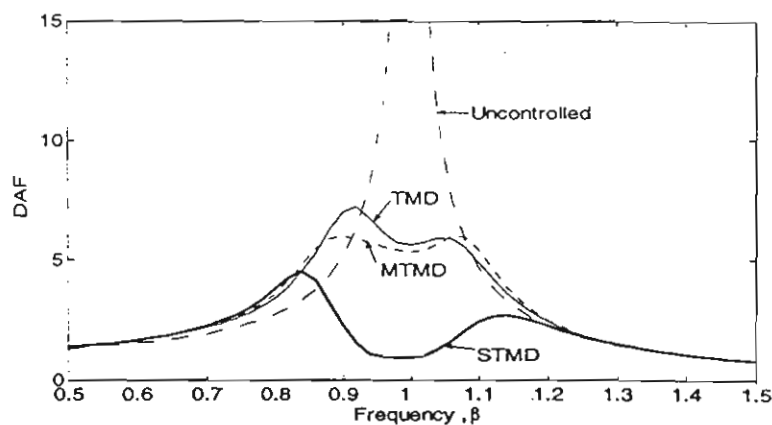
3.2 อิทธิพลของค่า R ต่อประสิทธิภาพของระบบ STMD

จากผลที่ได้จากการศึกษาเชิงตัวเลขจะพบว่าค่าการเปลี่ยนแปลงค่า R ในการสร้าง Control algorithm จะมีผลทำให้ response ของโครงสร้างในแบบ STMD เปลี่ยนไป (รูปที่ 3)



รูปที่ 3 แสดงผล Frequency-response ของการเลือกค่า R ใน Control algorithm

ซึ่งจากผลการศึกษพบว่าในระบบ STMD จะมีค่า R ที่เหมาะสมที่ทำให้ลดการสั่นของโครงสร้างอย่างมีประสิทธิภาพสูงสุดได้ ซึ่งแตกต่างจากในระบบแอคทีฟ ทั้งนี้เนื่องจากระบบ STMD มีข้อจำกัดของการให้แรงต่อระบบทำให้การเลือกค่า R ที่เหมาะสมต่อโครงสร้างจะเป็นปัจจัยที่สำคัญที่จะทำให้ประสิทธิภาพของระบบ STMD ค่อนข้างดี โดยในการศึกษานี้จะเลือกค่านี้นในการหาค่า R ที่เหมาะสมโดยให้ค่าพื้นที่ใต้กราฟของค่า DAF มีค่าต่ำที่สุด ซึ่งจะได้ว่าค่า R มีค่าประมาณ $10^{-9.5}$ การเลือกใช้ค่า R ที่เหมาะสมนี้มีความสำคัญมากเพราะเป็นการเลือก Algorithm ที่เหมาะสมและเป็นการใช้อุปกรณ์ที่มีอยู่อย่างคุ้มค่าที่สุด

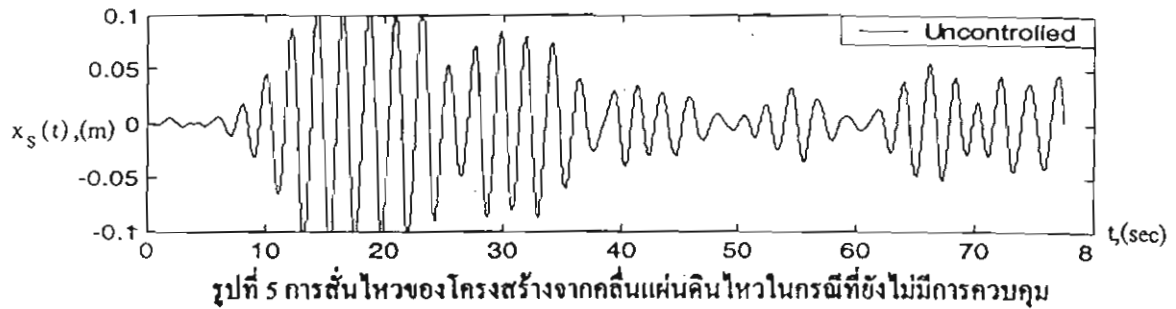


รูปที่ 4 แสดงผล Frequency-response curve ของระบบ Uncontrolled, TMD, MTMD และ STMD

จากรูปที่ 4 จะพบว่าในระบบ STMD ที่มีการออกแบบไว้อย่างเหมาะสมจะมีประสิทธิภาพสูงกว่าการควบคุมโดยวิธีการแบบ TMD และ MTMD เป็นอย่างมาก โดยพบว่าขนาดการสั่นของโครงสร้างลดลงได้มากและครอบคลุมช่วงความถี่ของแรงได้ในช่วงที่มากกว่าระบบ TMD และ MTMD

4. ตัวอย่างการศึกษาการควบคุมการสั่นไหวของโครงสร้างภายใต้แรงแผ่นดินไหว

เพื่อศึกษาและเปรียบเทียบประสิทธิภาพในการลดการสั่นไหวของโครงสร้างของระบบต่างๆ จึงได้ใช้โครงสร้างตัวอย่างมาศึกษาโดยใช้สัญญาณคลื่นแผ่นดินไหวที่วัดได้ในกรุงเทพมหานคร[2] จะได้ว่าโครงสร้างที่ยังไม่ได้มีการควบคุมการสั่นไหวจะได้ลักษณะการสั่นดังในรูปที่ 5

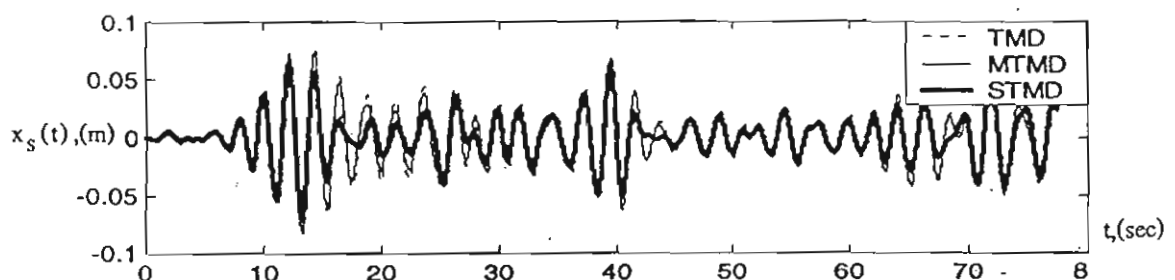


สำหรับกรณีการควบคุมโครงสร้างจะเปรียบเทียบวิธีการทั้งสามวิธีคือ TMD, MTMD และ STMD โดยค่าพารามิเตอร์ของแต่ละวิธีการสามารถแสดงได้ดังในตารางที่ 2

ตารางที่ 2 แสดงค่าพารามิเตอร์ต่างๆที่ใช้ในการคำนวณ

TMD	:	$m_d = 14.10 \text{ tons}$ $k_d = 111.2 \text{ kN/m}$ $c_d = 8.24 \text{ kN/(m/s)}$
MTMD	:	จำนวน TMDs = 5 (มวลรวมเท่ากับ 14.10 tons) ช่วงความถี่ = 15 % ของความถี่โครงสร้าง ค่าเฉลี่ยความถี่ของ TMDs = 0.957 เท่าของความถี่โครงสร้าง
STMD	:	$R = 1 \times 10^{-6.95}$ $c_{d, \min} = 0.05 \text{ kN/(m/s)}$ $c_{d, \max} = 79.14 \text{ kN/(m/s)}$ (Damping ratio = 1.00)

จากการใช้การควบคุมด้วยวิธีการแบบต่างๆ เมื่อนำมาคำนวณการสั่นไหวของโครงสร้างจะพบว่าการสั่นไหวมีขนาดลดลงโดยสามารถแสดงได้ดังในรูปที่ 6



สามารถเปรียบเทียบประสิทธิภาพของวิธีการควบคุมแบบต่างๆ ได้จากตารางที่ 3 ซึ่งเป็นตารางสรุปผลที่ได้ในรูปที่ 6 ซึ่งจะพบว่าวิธีการลดการสั่นไหวโดยระบบ STMD สามารถลดการสั่นไหวได้ดีกว่าทั้ง TMD และ MTMD

ตารางที่ 3 สรุปขนาดของการสั่นไหวของโครงสร้างจากคลื่นแผ่นดินไหวเปรียบเทียบกับกรณีต่างๆ

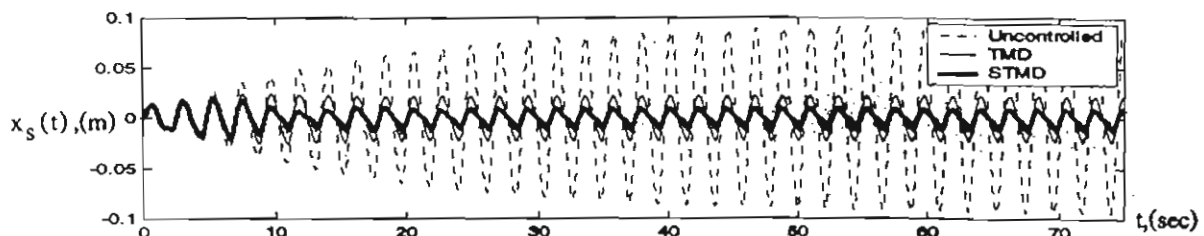
	Maximum		Root Mean Square	
	Displacement (ม.)	Normalized (%)	Displacement (ม.)	Normalized (%)
Uncontrolled	0.1098	100.00	0.0405	100.00
TMD	0.0835	76.05	0.0246	60.79
MTMD	0.0814	74.12	0.0240	59.28
STMD	0.0707	64.26	0.0200	49.56

5. ตัวอย่างการศึกษาการควบคุมการสั่นไหวของโครงสร้างภายใต้แรงลม

นอกจากระบบ STMD จะทำงานได้ดีเมื่อเทียบกับระบบอื่นๆ ในกรณีที่โครงสร้างสั่นเนื่องจากแผ่นดินไหวแล้ว ยังพิจารณากรณีที่มีแรงลมมากระทำต่อโครงสร้างดังสมการ 6 [1]

$$F_w = 3p\sin(\omega t) + 7p\sin(2\omega t) + 5p\sin(3\omega t) + 4p\sin(4\omega t) ; p=5000 \text{ kN} \quad (6)$$

โดยที่ ω เป็นความเร็วเชิงมุมของโครงสร้าง ผลของการสั่นไหวของโครงสร้างเมื่อรับแรงลมในกรณีที่ไม่มีระบบควบคุม, มีระบบควบคุมแบบ TMD และ STMD แสดงในรูปที่ 7 ซึ่งแสดงให้เห็นอย่างชัดเจนว่า STMD มีประสิทธิภาพสูงกว่าระบบ TMD มาก โดยที่ระบบ TMD และ STMD ลดการสั่นไหว 76 และ 89 % ตามลำดับ



รูปที่ 7 แสดงการสั่นไหวของโครงสร้างเนื่องจากแรงลมภายใต้การควบคุมแบบต่างๆ

6. บทสรุป

ในบทความนี้ได้นำเสนอการออกแบบระบบ STMD ให้มีประสิทธิภาพสูงสุดได้โดยการเลือกค่าความสามารถในการปรับค่าความหน่วงของตัวหน่วงในระบบ STMD ให้มีช่วงในการปรับที่สูง และการเลือกใช้ค่า R ที่เหมาะสม โดยคำนวณจากพื้นที่ใต้กราฟของ Frequency-response ให้มีค่าต่ำสุด ระบบ STMD ที่ออกแบบการควบคุมตามที่เสนอพบว่ามีประสิทธิภาพสูงกว่าทั้งระบบ TMD และ MTMD มากทั้งในกรณีที่โครงสร้างรับคลื่นแผ่นดินไหวและแรงลม

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การป้องกันความเสียหายของอาคารสูงเมื่อรับแรงแผ่นดินไหว ด้วยมวลหน่วงปรับค่า
DAMAGE PROTECTION OF HIGH-RISE BUILDINGS SUBJECTED TO EARTHQUAKE
BY TUNED MASS DAMPERS

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บทคัดย่อ: แผ่นดินไหวก่อให้เกิดความสูญเสียเป็นจำนวนมาก เช่น เม็กซิโก ปี 1985, ตุรกี และไต้หวัน ปี 1999 สำหรับประเทศไทย แม้ไม่ได้อยู่บริเวณที่มีความเสี่ยงภัยสูง แต่ก็ควรมีการศึกษาและเตรียมการป้องกันไว้ โดยเฉพาะการพังทลายของอาคารขนาดใหญ่ ซึ่งเป็นสาเหตุหนึ่งที่ทำให้เกิดการสูญเสียชีวิตและทรัพย์สินเป็นจำนวนมาก

งานวิจัยนี้เป็นการศึกษาถึงความเป็นไปได้ในการใช้มวลหน่วงปรับค่า (Tuned Mass Damper) เพื่อป้องกันความเสียหายของอาคารคอนกรีตเสริมเหล็กที่ไม่ได้ออกแบบเพื่อต้านทานแผ่นดินไหว โดยใช้คอมพิวเตอร์โปรแกรม IDARC ซึ่งสามารถวิเคราะห์พฤติกรรมของโครงสร้างในช่วงไม่ยืดหยุ่นได้ ผลการศึกษาพบว่าประสิทธิภาพของมวลหน่วงปรับค่า จะลดต่ำลงเมื่อความรุนแรงของแผ่นดินไหวมีมากขึ้น ทั้งนี้เนื่องมาจากการสูญเสียความสอดคล้องทางความถี่ของมวลหน่วงปรับค่า และการเพิ่มขึ้นของคุณสมบัติ hysteresis ของโครงสร้าง

ABSTRACT: Earthquake always follows with serious losses of lives or properties such as in Mexico (1985), Turkey and Taiwan (1999). Although Thailand is located in low seismicity area, it is worth to investigate and prepare, especially the preventive measure for high-rise building failure.

This research study considers the feasibility of using Tuned Mass Damper (TMD) for damage protection of a reinforced concrete building designed only for gravity loads. The IDARC program is employed to perform the inelastic analysis. The obtained results indicate that tuned mass damper effectiveness is decreased as the earthquake intensity increased. This is because the detuning effect and the hysteresis property of structure.

KEYWORDS: Tuned Mass Damper, Damage Analysis, R/C Building

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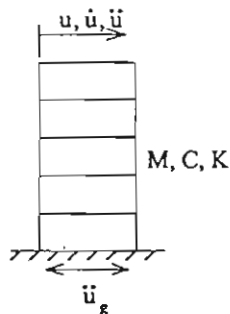
1. บทนำ

ในปัจจุบัน แผ่นดินไหวก่อให้เกิดความเสียหายต่อชีวิตและทรัพย์สินเป็นจำนวนมาก โดยเฉพาะเนื่องจากการพังทลายของอาคารและสิ่งก่อสร้างต่างๆ สำหรับประเทศไทยถึงแม้ยังไม่เคยเกิดแผ่นดินไหวขนาดรุนแรงในอดีต แต่มีโอกาสที่จะได้รับผลกระทบจากแผ่นดินไหวระยะไกล โดยเฉพาะกรุงเทพมหานคร ที่มีลักษณะชั้นดินเป็นดินอ่อน ซึ่งมีผลในการขยายขนาดของคลื่น ดังแผ่นดินไหวที่เคยเกิดขึ้นที่กรุงเม็กซิโกปี 1985

งานวิจัยนี้ทำการศึกษาอาคารคอนกรีตเสริมเหล็กความสูง 20 ชั้น ออกแบบตาม ACI-1995 ไม่คิดผลของแรงแผ่นดินไหว ถูกกระทำภายใต้คลื่นแผ่นดินไหวที่เม็กซิโก (SCT S00E) และคลื่นที่วัดได้จากอาคารไบฮอก ในปี 1995 [1] การวิเคราะห์เป็นแบบไม่ยืดหยุ่น (Inelastic Analysis) โดยใช้คอมพิวเตอร์โปรแกรม IDARC [2] วิเคราะห์และได้ปรับปรุงอาคารดังกล่าวด้วยมวลหน่วงปรับค่า (Tuned Mass Damper)

2. การวิเคราะห์โครงสร้างภายใต้แรงเนื่องจากแผ่นดินไหว

2.1 การวิเคราะห์โครงสร้างที่มีพฤติกรรมแบบยืดหยุ่น โดยคิดว่าค่าสติเฟนสของโครงสร้างมีค่าคงที่



$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = -M\ddot{u}_g \quad (1)$$

โดย M , C และ K เป็นมวล, ความหน่วง และสติเฟนสของโครงสร้าง ตามลำดับและ $\ddot{u}$, $\dot{u}$ และ u เป็นความเร่ง, ความเร็ว และการเปลี่ยนตำแหน่งของโครงสร้าง $\ddot{u}_g$ เป็นความเร่งที่ฐาน

รูปที่ 1 แบบจำลองโครงสร้าง

2.2 การวิเคราะห์โครงสร้างที่มีพฤติกรรมแบบไม่ยืดหยุ่น ค่าสติเฟนสของโครงสร้างมีการเปลี่ยนแปลงเนื่องจากความเสียหายที่เกิดขึ้น

$$M\ddot{u}(t) + C\dot{u}(t) + f_s(u, \dot{u}) = -M\ddot{u}_g \quad (2)$$

โดยที่ $f_s(u, \dot{u})$ เป็นแรงสติเฟนส มีค่าเปลี่ยนแปลงตามค่าและทิศทางของการเปลี่ยนตำแหน่ง

3. การวิเคราะห์ความเสียหายของโครงสร้างคอนกรีตเสริมเหล็กภายใต้แรงแผ่นดินไหว

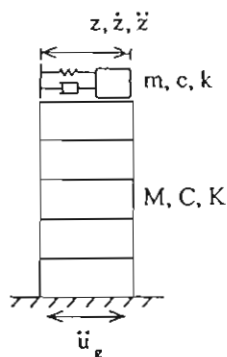
ในงานวิจัยนี้ใช้วิธีการวิเคราะห์ความเสียหายที่เสนอโดย Park และ Ang [3] ซึ่งอยู่ในรูปของดัชนีความเสียหาย (Damage Index)

$$DI = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_y \delta_u} \int dE \quad (3)$$

โดยที่ DI เป็นดัชนีความเสียหาย, δ_m เป็นการเปลี่ยนตำแหน่งสูงสุดภายใต้แผ่นดินไหว, δ_u เป็นการเปลี่ยนตำแหน่งประลัย, Q_y เป็นกำลังครากของชิ้นส่วน, β เป็นค่าคงที่ และ $\int dE$ เป็นพลังงานสะสมของชิ้นส่วน

ค่าดัชนีความเสียหายที่ได้จะมีค่าอยู่ระหว่าง 0 ถึง 1 โดยจะมีค่ามากหากเกิดความเสียหายมาก

4. มวลหน่วงปรับค่า (Tuned Mass Damper)



การใช้มวลหน่วงปรับค่าในอาคารมีวัตถุประสงค์เพื่อลดปริมาณการสลายพลังงานของโครงสร้างอันเนื่องมาจากแผ่นดินไหว ซึ่งส่งผลให้ความเสียหายลดลง สมการการเคลื่อนที่ของโครงสร้างที่มีมวลหน่วง (รูปที่ 2) สามารถเขียนได้เป็น

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = c\dot{z}(t) + kz(t) - M\ddot{u}_g(t) \quad (4.1)$$

$$m\ddot{z}(t) + c\dot{z}(t) + kz(t) = -m\ddot{u}(t) - m\ddot{u}_g(t) \quad (4.2)$$

รูปที่ 2 โครงสร้างที่มีมวลหน่วงปรับค่า

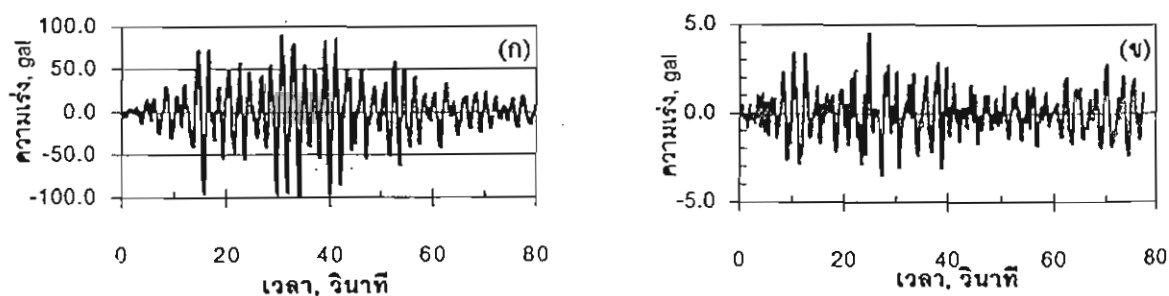
5. การวิเคราะห์โดยคอมพิวเตอร์โปรแกรม IDARC

โปรแกรม IDARC สามารถคำนวณการตอบสนองของอาคารคอนกรีตเสริมเหล็กที่มีพฤติกรรมแบบไม่ยืดหยุ่น โดยอธิบายพฤติกรรมการรับแรงของโครงสร้างด้วยความสัมพันธ์ของโมเมนต์กับความโค้งของหน้าตัดด้วยเส้นตรงสามส่วน (Tri-linear) ซึ่งจะพิจารณาการแตกร้าวของคอนกรีต, การครากของเหล็ก, การวิบัติของหน้าตัด ตติพจน์ของชิ้นส่วนคดผลของการคราก (Spread Plasticity Model และ Yield Penetration Model), ผลการเสื่อมลงของสติฟเนส (Stiffness Degradation), การสูญเสียกำลัง (Strength Deterioration) และการเกิดรอยร้าวและการครูดของเหล็กเสริม (Crack-closure/Bond-slip or Pinching) ภายใต้แรงวัฏจักร (Hysteretic Rules)

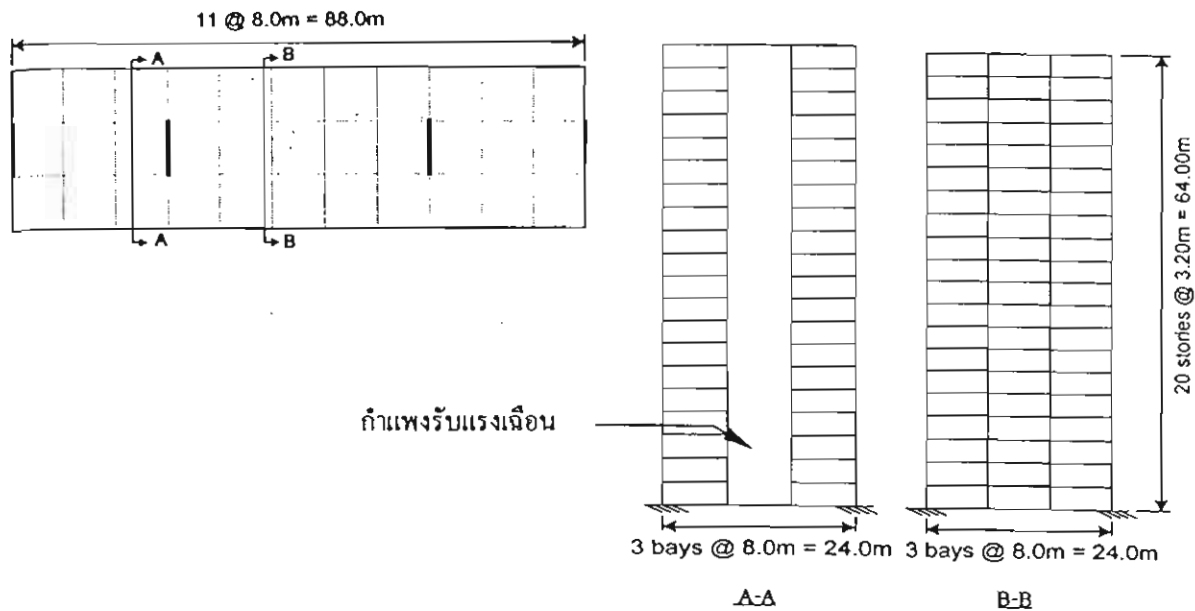
ในงานวิจัยนี้ได้จำลองมวลหน่วงปรับค่าเป็นโครงหักฉาก (Portal Frame) ที่ตำแหน่งบนสุดของอาคารและมีมวลเป็นครึ่งหนึ่งของชั้นบน คำนวณความถี่ธรรมชาติและอัตราส่วนความหน่วงจาก Warburton [4] เนื่องจากโปรแกรม IDARC คำนวณผลของอัตราส่วนความหน่วงทั้งโครงสร้าง ดังนั้นตัวหน่วงที่ใส่ในมวลหน่วงซึ่งเป็น Viscous-elastic Damper จำเป็นต้องปรับค่าเพื่อให้ได้ค่าความหน่วงเท่ากับค่าที่ต้องการ

6. กรณีศึกษากับอาคารคอนกรีตเสริมเหล็กความสูง 20 ชั้น

อาคารที่ศึกษาเป็นอาคารคอนกรีตเสริมเหล็กความสูง 20 ชั้น (รูปที่ 4) ออกแบบตาม ACI-1995 และ ASCE 7-95 ใช้กำลังคอนกรีต 300 กก/ซม² เหล็กเสริม SD40 และ SR24 ใช้คลื่นแผ่นดินไหวที่เม็กซิโก ปี 1985 (SCT S00E) ซึ่งเป็นคลื่นที่บันทึกได้บนชั้นดินอ่อน และคลื่นที่บันทึกได้ใช้อาคารโบหยก (BYK) (รูปที่ 3)



รูปที่ 3 คลื่นแผ่นดินไหว บันทึกได้จาก (ก) เม็กซิโก (SCT S00E) และ (ข) ใช้อาคารโบหยก (BYK)



รูปที่ 4 รูปแปลนและรูปคั้งของกรณีศึกษา

7. พฤติกรรมของอาคารภายใต้แรงเนื่องจากแผ่นดินไหวและผลของมวลหน่วงปรับค่า

จากโปรแกรม IDARC พบว่าความถี่ธรรมชาติของอาคารที่ศึกษามีค่า 0.47 Hz ซึ่งเป็นค่าที่ใกล้เคียงกับความถี่เด่นชัด (Predominant Frequency) ของแผ่นดินไหวที่ใช้ ภายใต้ขนาดความเร่งของแผ่นดินไหวเท่ากับ 0.001g การเปลี่ยนตำแหน่งที่ชั้นบนแสดงดังรูปที่ 5 โดยที่ไม่มีความเสียหายเกิดขึ้นกับโครงสร้าง ขณะที่ขนาดความเร่ง 0.098g การเปลี่ยนตำแหน่งที่ชั้นบนมีค่ามากขึ้นและมีการเปลี่ยนตำแหน่งถาวรภายหลังจากแผ่นดินไหว อันเนื่องจากความเสียหายภายในองค์อาคาร ดังรูปที่ 6

การเปรียบเทียบการสั่นไหวของอาคารที่ติดตั้งและไม่ติดตั้งมวลหน่วงปรับค่าภายใต้แผ่นดินไหวที่มีขนาดความเร่ง 0.001g และ 0.098g แสดงในรูปที่ 5 และ 6 ตามลำดับ จะเห็นได้ว่าผลของมวลหน่วงปรับค่าในการลดการสั่นไหวของอาคารแทบไม่สามารถสังเกตเห็นได้ในกรณีที่มีขนาดความเร่ง (0.098g) ซึ่งในกรณีนี้หากพิจารณาถึงความเสียหายในอาคารทั้งในส่วนของการคานและเสาจะดังรูปที่ 7 ซึ่งจากรูปพบว่าคลื่นแผ่นดินไหวทั้งกรณี SCT และ BYK ก่อให้เกิดความเสียหายที่คล้ายคลึงกัน โดยพบความเสียหายในชั้นที่ 8-20 ในส่วนของคานและความเสียหายอย่างมากในส่วนของเสาและกำแพงรับแรงเฉือนบริเวณชั้นล่าง นอกจากนี้ยังพบว่ามวลหน่วงปรับค่าไม่สามารถลดความเสียหายของอาคารได้ ทั้งนี้เนื่องจากความเสียหายขององค์อาคารที่ทำให้จังหวะการสั่นไหวของอาคารเปลี่ยนแปลงไป และส่งผลให้มวลหน่วงปรับค่าไม่สามารถทำงานได้อย่างมีประสิทธิภาพ

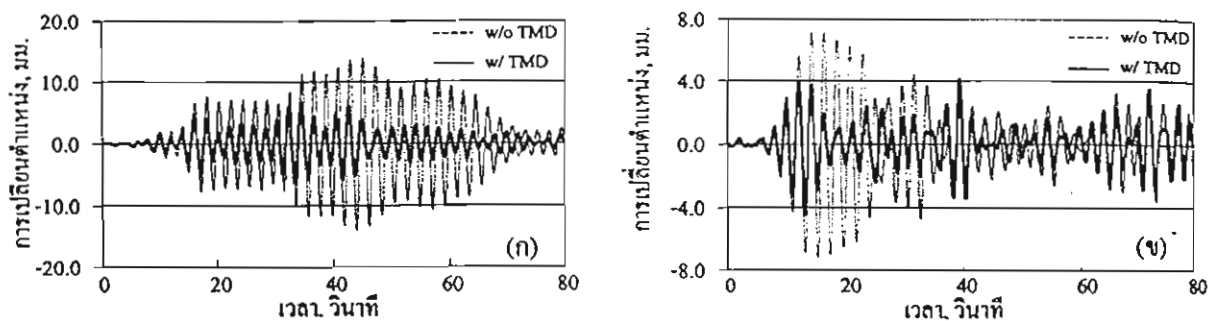
ผลของขนาดความเร่งของคลื่นแผ่นดินไหว (Peak Ground Acceleration, PGA.) ต่อพฤติกรรมการสั่นไหวแสดงในรูปที่ 8 ในเทอมของ root-mean-square (RMS) ของการเคลื่อนที่ที่ชั้นบนของอาคาร จะเห็นได้ว่ามวลหน่วงปรับค่าสามารถลดการสั่นไหวให้กับอาคารได้ในช่วงขนาดความเร่งต่ำ โดยที่ขนาดความเร่ง 0.001g สามารถลด RMS ของการเคลื่อนที่ที่ชั้นบนได้ 52.0% และ 38.0% สำหรับ SCT และ BYK ตามลำดับ แต่ในกรณีที่มีขนาดความเร่งสูง จะพบว่า RMS ของการเคลื่อนที่ที่ชั้นบนของอาคารที่ติดตั้งและไม่ติดตั้งมวลหน่วงปรับมีค่าแตกต่างกันน้อยมาก

เช่นที่ขนาดความเร่ง 0.098g มวลหน่วงปรับค่าสามารถลด RMS ของการเคลื่อนที่ชั้นบนได้เพียง 5.5% และ 3.0% สำหรับ SCT และ BYK ตามลำดับ

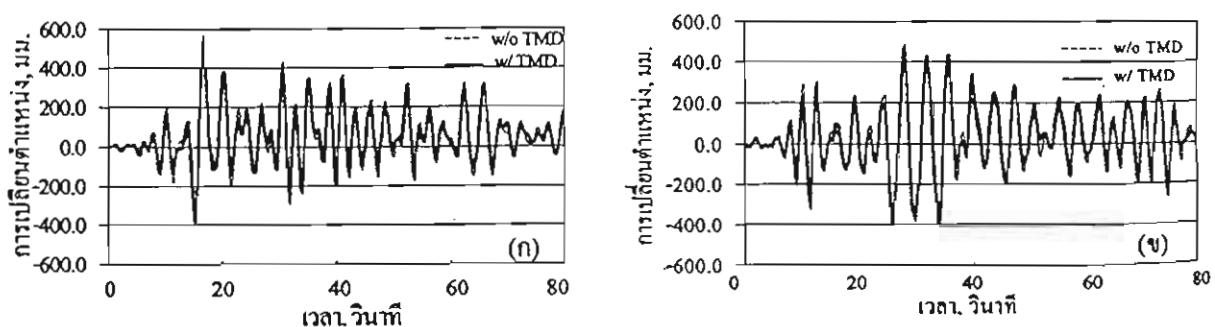
รูปที่ 9 แสดงผลของขนาดความเร่งของแผ่นดินไหวต่อความเสียหายของอาคาร ซึ่งพบว่าความเสียหายของอาคารเพิ่มขึ้นตามขนาดความเร่งที่เพิ่มขึ้น นอกจากนี้ยังแสดงผลของมวลหน่วงปรับค่าว่าไม่สามารถลดความเสียหายของอาคารได้อย่างมีนัยสำคัญ โดยที่ขนาดความเร่ง 0.098g สามารถลดความเสียหายได้ 14.4% และ 13.3% สำหรับ SCT และ BYK ตามลำดับ

8. สรุป

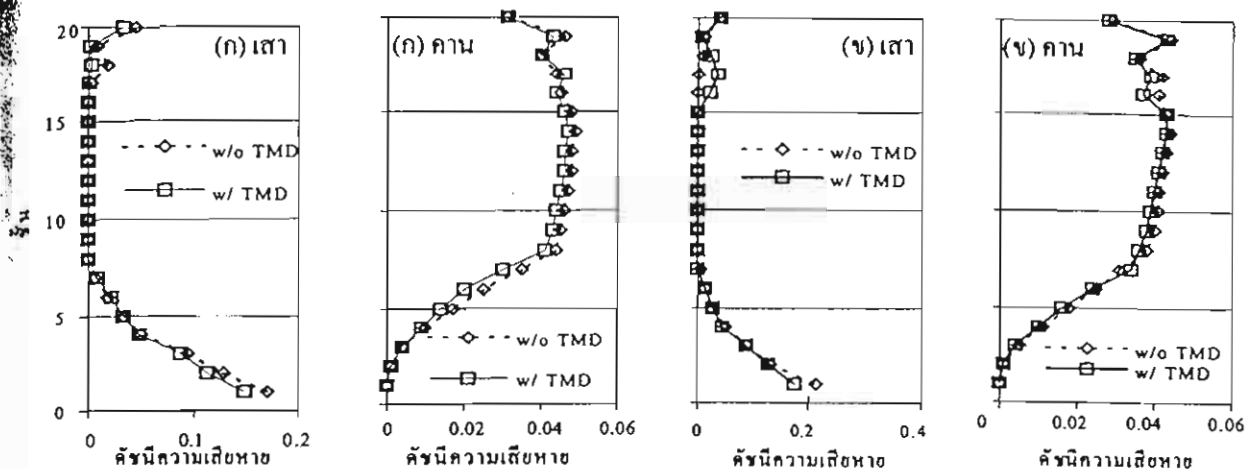
ภายใต้คลื่นแผ่นดินไหว SCT และ BYK พบว่าจากกรณีศึกษาอาคารคอนกรีตเสริมเหล็กความสูง 20 ชั้นมีพฤติกรรมเป็นแบบเสาแข็งแรง-คานอ่อน (Strong Column-weak Beam) เนื่องจากมีการครากของคานทั่วทั้งอาคาร แต่อย่างไรก็ตามก็มีการครากของกำแพงรับแรงเฉือนและเสาที่บริเวณชั้นล่างด้วย สำหรับมวลหน่วงปรับค่านั้นจะมีประสิทธิภาพในการลดการตอบสนองของอาคารได้ดีก็ต่อเมื่อแผ่นดินไหวมีขนาดความเร่งต่ำๆ เท่านั้น และประสิทธิภาพจะลดลงเมื่อขนาดความเร่งมีค่าสูงขึ้น อันเป็นผลมาจากการสูญเสียความสอดคล้องทางความถี่และการเพิ่มขึ้นของการสลายพลังงานเนื่องจาก hysteresis



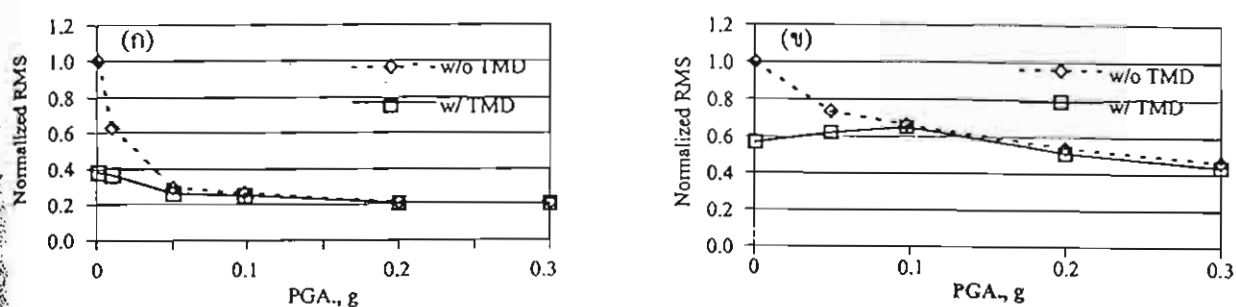
รูปที่ 5 การเปลี่ยนตำแหน่งบริเวณชั้นบนภายใต้แรงแผ่นดินไหวขนาด 0.001g (ก) กรณี SCT (ข) กรณี BYK



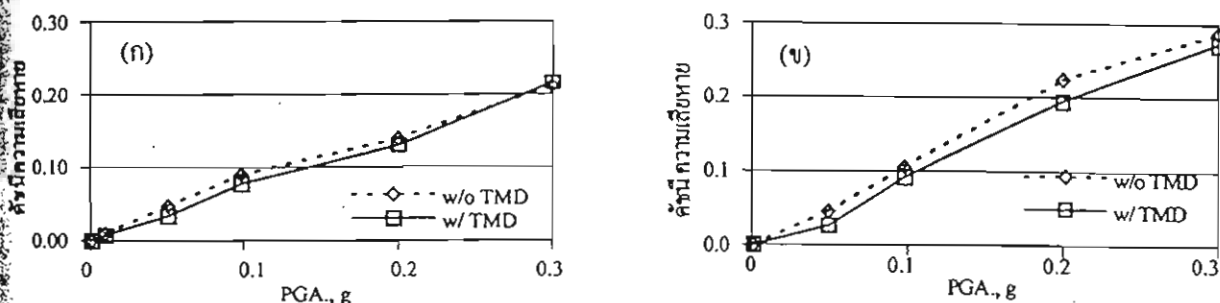
รูปที่ 6 การเปลี่ยนตำแหน่งบริเวณชั้นบนภายใต้แรงแผ่นดินไหวขนาด 0.098g (ก) กรณี SCT (ข) กรณี BYK



รูปที่ 7 ความเสียหายของอาคารภายใต้แรงแผ่นดินไหวขนาด 0.098g (ก) SCT (ข) BYK



รูปที่ 8 RMS ของการเคลื่อนที่ชั้นบน (ก) กรณี SCT (ข) กรณี BYK



รูปที่ 9 ความเสียหายของอาคาร (ก) กรณี SCT (ข) กรณี BYK

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ภาคผนวก จ

การจดทะเบียนสิทธิบัตร



สถาบันทรัพย์สินทางปัญญาแห่งจุฬาลงกรณ์มหาวิทยาลัย
Chulalongkorn University Intellectual Property Institute

ที่ สทปจ. 142 /2545

๒ มีนาคม 2545

เรื่อง ขอความเห็น

เรียน ศาสตราจารย์ ดร.ปณิธาน ลักคุณะประสิทธิ์

- สิ่งที่ส่งมาด้วย 1.สำเนาหนังสือที่ พณ 0706/2048 ลงวันที่ 18 กุมภาพันธ์ 2545 เรื่องการชำระค่าธรรมเนียม
ประกาศโฆษณาคำขอรับสิทธิบัตร
2.ข้อโต้สิทธิ และบทสรุปการประดิษฐ์ คำขอรับสิทธิบัตรเรื่อง คลิปยืดขาของอเหล็กปลอก

ตามที่สถาบันทรัพย์สินทางปัญญาแห่งจุฬาลงกรณ์มหาวิทยาลัย ได้รับมอบหมายจากท่าน
อาจารย์ ให้ดำเนินการยื่นคำขอรับสิทธิบัตรการประดิษฐ์เรื่อง คลิปยืดขาของอเหล็กปลอก นั้น ทางสถาบันฯ ได้ขอ
เรียนให้ทราบว่า งานประดิษฐ์ดังกล่าวได้มีคำสั่งให้ชำระค่าธรรมเนียมประกาศโฆษณา ภายใน 60 วัน นับแต่วันที่ได
รับหนังสือคำสั่ง

จึงเรียนมาเพื่อโปรดพิจารณา และหากประสงค์จะให้ทางสถาบันฯ ดำเนินการต่อไป ขอให้โปรดส่ง
ค่าแปลของข้อโต้สิทธิ และบทสรุปการประดิษฐ์ เป็นภาษาอังกฤษ และแจ้งเป็นลายลักษณ์อักษรมายังสถาบันฯ ภายใน
ในวันที่ 5 เมษายน 2545 ด้วย จักขอบพระคุณยิ่ง

ขอแสดงความนับถือ

(รองศาสตราจารย์ สุธรรม อยู่ในธรรม)

ผู้อำนวยการใหญ่

ข้อถือสิทธิ

1. คลิปยัดขาของอเหล็กปลอกที่มีลักษณะเฉพาะคือ ประกอบด้วยคลิปยัดขาของอของเหล็ก
ปลอกที่มีของอระหว่าง 80-100 องศา สำหรับใช้ในเสาคอนกรีตเสริมเหล็กซึ่งจะทำให้เสาคอนกรีต
เสริมเหล็กที่รัดด้วยเหล็กปลอกที่มีของอระหว่าง 80-100 องศา มีพฤติกรรมความเหนียวดีขึ้นทั้งใน
5 ด้านการรับแรงในแนวแกน หรือแรงในแนวแกนร่วมกันแรงดันและแรงเฉือน

2. คลิปยัดขาของอเหล็กปลอกตามข้อถือสิทธิข้อ 1 ซึ่งเพิ่มตะแกรงหรือลวดตาข่ายทแยงหน้า
รอบนอกโครงเหล็กเสริมเสาคอนกรีตเสริมเหล็กในกรณีที่ต้องการพฤติกรรมความเหนียวของเสาเพิ่ม
ขึ้น

3. คลิปยัดขาของอเหล็กปลอกตามข้อถือสิทธิข้อ 1 ที่นำมาตัดให้มีลักษณะเป็นตะขอที่ปลาย
10 สองข้างแล้วจัดเป็นรูปวีโดยมนที่ยอด ปลายที่เป็นตะขอให้เกิดรอยยัดขาของอเหล็กปลอก

4. ตะแกรงเหล็กตามข้อถือสิทธิข้อ 2 ทำด้วยลวดที่ใช้ในงานก่อสร้างเป็นตาข่ายในแนวทแยง
มุม หรืออาจทำด้วยตะแกรงเหล็กสำเร็จรูปก็ได้ ตะแกรงเหล็กใช้พันรอบนอกเหล็กเสริมเสาคอนกรีต

บทสรุปการประดิษฐ์

คลิปปิดขาของอเหล็กปลอก มีลักษณะเป็นเหล็กงอโค้งคล้ายตะขอใช้ยึดขาของอเหล็กปลอกที่มีของระหว่าง 80-100 องศาในเสาคอนกรีตเสริมเหล็กสำหรับการใช้งานทั่วไป และเพิ่มตะแกรงเหล็กทแยงพันรอบนอกโครงเหล็กเสริมของเสาคอนกรีตเสริมเหล็ก สำหรับกรณีใช้งานที่ต้องการพฤติกรรมความเหนียวมากเป็นพิเศษทางด้านแรงดัดร่วมกับแรงเฉือน

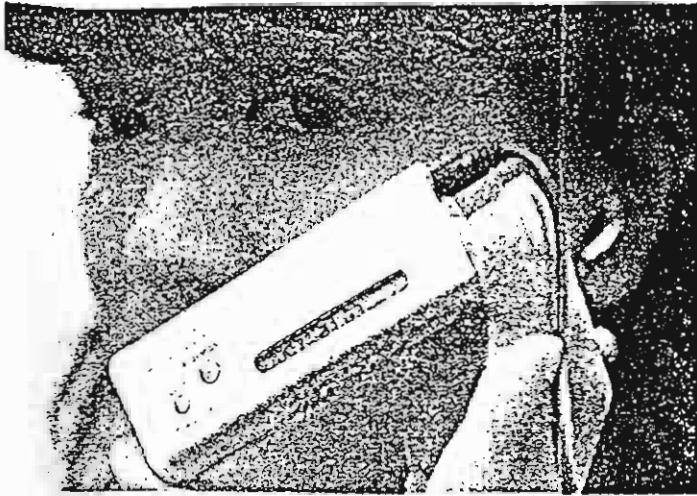
ภาคผนวก ช

ผลงานวิจัยเกี่ยวกับ Hook-Clip ได้รับการเผยแพร่

ภาคผนวก ซ

รางวัล

วอลท์แมนจิ๋ว



พนักงานบริษัทโซนี่ โชว์เครื่องเล่นวอลท์แมนรุ่น "เอ็นดับเบิลยู-อี 3" ซึ่ง "วอลท์แมนรุ่นใหม่ล่าสุดของบริษัท ที่มีลักษณะพิเศษอยู่ที่ขนาดเล็กกะทัดรัด และมีน้ำหนักเพียง 45 กรัม หรือเท่ากับเหรียญ 1 บาท สามารถบันทึกเพลงได้นานถึง 80 นาทีจากแหล่งที่เป็นอิเล็กทรอนิกส์ชนิดนี้ โดยกำหนดวางตลาดในเดือนมิถุนายนนี้ (เอที)

‘ยาอี’ตัวการความจำเสื่อม

สำนักข่าวต่างประเทศรายงานว่า ดร.ฟิลิป เมอร์ฟี และคณะนักวิจัยของสถาบันเอดจ์ ฮิลล์ คอลเลจ ออฟ ไฮเออร์ เอ็ดดูเคชั่น ที่เมืองออร์มสเบิร์ก ได้เปิดเผยผลวิจัยล่าสุด ที่พบว่า ยาอีจะเข้าไปทำลายความจำของสมอง โดยผู้ที่เสพยาอี และผู้ที่เคยเสพยาอี จะทำคะแนนจากแบบทดสอบความจำได้ต่ำเมื่อเปรียบเทียบกับผู้ที่ไม่เสพยาอี ผลการวิจัยชิ้นนี้ตีพิมพ์ลงในวารสารจิตวิทยาอังกฤษ

โดยนักจิตวิทยาได้เปรียบเทียบกลุ่มทดลอง 3 กลุ่ม คือ กลุ่มที่ไม่เสพยาอี กลุ่มที่เสพยาอีเมื่อ 6 เดือนก่อน แต่ปัจจุบันเลิกแล้ว และกลุ่มที่กำลังเสพยาอี โดยให้แบบทดสอบความจำในการทำงาน เช่น การให้คำแนะนำของภาพที่เห็น การให้ฟัง

เสียง และการเข้าใจภาษา โดยกำหนดเวลาให้กับผู้ทดสอบทุกกลุ่ม ผลปรากฏว่า ผู้ที่เสพยาอี และผู้ที่เคยเสพยาอี เมื่อ 6 เดือนก่อน คอบแบบทดสอบผิดพลาดมากกว่าผู้ที่ไม่เคยเสพยาอี ซึ่งผลจากการทดลองบ่งบอกเป็นนัยว่า ยาอีน่าจะส่งผลในระยะยาวต่อผู้เสพยาอี และอาจจะไม่สามารถกลับมาเป็นปกติได้ ผู้ที่คิดเสพยาอีจะต้องคิดให้รอบคอบขึ้น และการจะต้องออกค่าเตือนเกี่ยวกับเรื่องนี้ด้วย

ดร.เมอร์ฟีกล่าวว่า อาการความจำเสื่อมอันเนื่องมาจากเสพยาอี อาจเนื่องมาจากเส้นประสาทในสมองที่บรรจุสารเซโรโตนิน หลังจากถูกยาอีทำลาย จะมีการสร้างใหม่ แต่เป็นการสร้างที่ผิดปกติ เพราะจากการตรวจกับสัตว์ทดลองก็พบความผิดปกติที่วุ่นวายจริงๆ

สภ.ประทีปฯ คลิปยึกขยาเหล็กปลอก

กินปลาน้อยต่อสัปดาห์
ขาดไขมัน ไม่อึดตัว
เสี่ยงเป็น โรคซึมเศร้า

สำนักข่าวต่างประเทศรายงานว่า ดร.แอนดี หันสตาแนม จากมหาวิทยาลัยควอเทียร์ ในฟิเนแลนด์ วิจัยพบว่า คนที่บริโภคปลาน้อยกว่าสัปดาห์ละครั้ง จะมีความเสี่ยงถึงร้อยละ 31 ที่จะมีการบริโภคโรคซึมเศร้ามากกว่าคนที่บริโภคปลาน้อยกว่าสัปดาห์ละครั้ง ทั้งนี้การวิจัยวิจัยพบว่าในเนื้อปลา มีกรดไขมันไม่อิ่มตัว ที่มีชื่อว่าโอเมก้า-3 แต่ก็ยังเร็วเกินไปที่จะแนะนำให้ประชาชนหันมากินปลา หรือบริโภคกรดไขมันไม่อิ่มตัว เพื่อหลีกเลี่ยงภาวะซึมเศร้า และแม้ว่าข้อมูลที่พบนี้พบแค่ในผู้หญิงเท่านั้น แต่ผู้วิจัยก็เชื่อว่า ผลลัพธ์ที่ได้ก็จะพบในผู้ชายเช่นกัน

ข่าวแจ้งว่า การศึกษาครั้งนี้ได้มาจากการค้นคว้าวิจัยในฟินแลนด์ โดยในฟินแลนด์ได้ศึกษาอาการซึมเศร้าและความถี่ในการกินอาหารจำพวกปลา จากการศึกษพบว่า ร้อยละ 30 กินปลาน้อยกว่าสัปดาห์ละครั้ง ซึ่งถือว่าอยู่ในปริมาณน้อย โดยร้อยละ 28 ของกลุ่มคนเหล่านี้มีอาการซึมเศร้าด้วยเช่นกัน

● รับกม.ออกแบบอาคารกันแผ่นดินไหว

นายปณิธาน ลักคุณะประสิทธิ์ เมธีวิจัยอาวุโส สำนักงานกองทุนสนับสนุนการวิจัย (สกว.) เปิดงานที่พื้นที่บางสวนทางภาคเหนือ ภาคตะวันออกเฉียงเหนือ และ จ.กาญจนบุรี มักจะเกิดเหตุแผ่นดินไหวบ่อยครั้ง เพราะพื้นที่เหล่านี้อยู่ในแนวรอยเลื่อนกลางเกิดแผ่นดินไหว และแม้ว่าแรงสั่นสะเทือนที่เกิดขึ้นแต่ละครั้งจะไม่มากนัก แต่ก็ยังส่งผลให้อาคารบ้านเรือนและสิ่งปลูกสร้างได้รับความเสียหายอย่างมาก เพราะโครงสร้าง อาคารบ้านเรือนเกือบทั้งหมดในประเทศไทยไม่ได้ทำขึ้นเพื่อป้องกันการเกิดแผ่นดินไหว คนจึงได้ประดิษฐ์คลิปยึกขยาเหล็กปลอก เพื่อช่วยให้เสาคอนกรีตเสริมเหล็กที่มีคลิปดังกล่าว สามารถทนต่อแรงโยกคลอนได้มากขึ้น

นายปณิธานกล่าวว่า ปกติในการก่อสร้างอาคารโดยทั่วไปเสาคอนกรีตเสริมเหล็กที่ใช้เป็นตัวรับน้ำหนักจะประกอบไปด้วยเหล็กเส้นที่ก่อตัวเป็นโครงเสา อันมีเหล็กเส้นที่จะดึงตรงขนานกับเสา และเหล็กปลอกที่มีปลายทั้งสองของเหล็กมาบรรจบกัน จะหักเป็นมุม 90 องศา ขนานไปกับทรงของเสาเหล็กปลอก ซึ่งโครงสร้างนี้เรียกว่า หากมีแรงกดมาทางด้านบนหรือด้านข้าง เช่น เมื่อเกิดเหตุแผ่นดินไหว โอกาสที่เหล็กปลอกจะหักตัว หรือบิด

ออกเป็นไปได้อย่างสูง ซึ่งจะทำให้เสาไม่สามารถรับน้ำหนักได้ อาคารหลังดังกล่าวจึงมีโยกคลอนหลายครั้ง แต่หากก่อนก่อสร้างมีการใช้คลิปยึกขยาเหล็กปลอก อุปกรณ์ดังกล่าวจะเข้าไปยึดช่วงรอยต่อของเหล็กปลอกที่มุมของเสา โดยจะช่วยชะลอการแอ่นของเหล็กปลอก ทำให้แกนเสาถอยกลับและเหล็กยึดในเสาคงรูปได้นานขึ้น ผลก็คือ เสาสามารถทนการโยกตัวไปมาจากแรงดันข้าง เช่น แรงที่เกิดจากแผ่นดินไหว และแรงลม ได้มากกว่าเสาที่ไม่มีคลิปยึกขยาเหล็กปลอก

เมื่อถามว่า คลิปเหล็กดังกล่าวทำมาจากเหล็กประเภทใด และเมื่อใช้กับอาคารบ้านเรือนแล้วจะทนต่อแรงแผ่นดินไหวได้กี่ริกเตอร์ นายปณิธานกล่าวว่า คลิปเหล็กดังกล่าวทำมาจากเหล็กเหนียวธรรมดา แต่ได้ออกแบบโครงสร้าง และกำหนดขนาดการยึดติดกับตัวโครงเหล็กปลอก ไม่สามารถถอดได้ถ้าเมื่อใช้คลิปตัวนี้แล้วตัวอาคารจะสามารถรับการสั่นไหวได้กี่ริกเตอร์ ทั้งนี้ขึ้นอยู่กับส่วนประกอบหลักอย่างอื่นด้วย อย่างไรก็ตาม อุปกรณ์ตัวนี้ประดิษฐ์ขึ้นมาเพื่อรองรับกฎหมายอาคารข้อกำหนดอาคารเพื่อป้องกันแผ่นดินไหว ของกระทรวงมหาดไทยด้วย

ติดเครื่องส่งสัญญาณเตือนภัยล่วงหน้า

Journal of Management Inquiry

สร้างกลียัตเหล็กปลอกถนนแรกของโลก

ศ.ดร.ปณิธาน ลักคุณะประสิทธิ์

๑๕๗ หัวหน้าภาควิชา วิศวกรรมโครงสร้าง จุฬาลงกรณ์มหาวิทยาลัย
สัมภาษณ์



ศ.ดร.ปณิธาน ลักคุณะประสิทธิ์
หัวหน้าภาควิชา วิศวกรรมโครงสร้าง
จุฬาลงกรณ์มหาวิทยาลัย ผลงานคิดค้นของเขา
กลายเป็นโครงการวิจัยที่มีความปลอดภัย
จากแผ่นดินไหวได้อีกระดับหนึ่ง

แม้ว่าประเทศไทยจะไม่อยู่บนโซนเขตอันตรายที่จะเกิดแผ่นดินไหวรุนแรง แต่ก็ไม่สามารถมองข้ามภัยจากแผ่นดินไหวได้ ในประเทศที่มีแผ่นดินไหวรุนแรง โดยปกติจะมีระเบียบกฎหมายบังคับไว้ให้ใช้เหล็กปลอก 135 องศาในงานก่อสร้างเนื่องจากพฤติกรรมของเหล็กปลอก 135 องศา จะดีกว่าเหล็กปลอก 90 องศา

แต่การนำเหล็กมาดัดเพื่อให้ได้ 135 องศา นั้นทำได้ยาก เพราะว่าถ้าเหล็กมันวางผิดตำแหน่งไป เหล็กปลอกนี้จะใส่เข้ายากหรือบางทีใส่ไม่ได้เลย ฉะนั้นในหลายประเทศจึงเลือกใช้เพียงเหล็กปลอก 90 องศา ซึ่งจะทนต่อการโยกได้ไม่น้อยส่งผลให้สิ่งก่อสร้างนั้นพังง่ายขึ้นเมื่อเวลาที่เกิดแผ่นดินไหว

ศ.ดร.ปณิธาน ลักคุณะประสิทธิ์ หัวหน้าภาควิชา วิศวกรรมโครงสร้าง จุฬาลงกรณ์มหาวิทยาลัย ผู้เชี่ยวชาญด้านวิศวกรรมโครงสร้างและศึกษาวิจัยเรื่องราวการเกิดแผ่นดินไหว จึงได้คิดค้นประดิษฐ์กลียัตเหล็กปลอก 90 องศาขึ้น เพื่อให้พฤติกรรมของเหล็กปลอก 90 องศา เทียบเท่ากับเหล็กปลอก 135 องศา

"ที่มาของความคิดนี้เกิดขึ้นในขณะที่ประชุมวิศวกรรมแผ่นดินไหวของ World Conference on Earthquake Engineering ที่เม็กซิโก ซึ่งในเวลานั้นเพิ่งเกิดแผ่นดินไหวที่โกเบได้ไม่นาน มีวิศวกรมาฉายภาพให้ดูเกี่ยวกับความเสียหายที่เกิดขึ้น ก็มีลักษณะให้เห็นว่าเสาแตก เหล็กปลอกอ้าเยอะเยอะ ระหว่างนั้นยังอยู่เห็นมีเอกสารที่มีกลีบกระดาศ ก็เกิดความคิดว่าทำไมเราไม่เอากลีบแบบนี้มากลีเหล็กปลอกดูบ้าง พอกลับมาก็เลยให้นักเรียนลองทำดู ปรากฏว่าได้ผล แต่วามันยากกว่ากลีบกระดาศ เพราะเรากลีจากข้างนอก"

โดยได้ใช้เวลาในการทดลองนาน 2 ปี ถึงมั่นใจว่ากลียัตเหล็กปลอก 90 องศาซึ่งทำจากเหล็กรรณดาสำเร็จรูป ทำงานได้ผลเทียบเท่ากับเหล็กปลอก 135 องศา แต่ทำงานได้ง่าย สะดวกและรวดเร็วกว่า กล่าวคือสามารถติดตั้งโดยใช้เวลาเพียง 10 วินาทีต่อตัวเท่านั้น ในขณะที่เดียวกันต้นทุนที่ใช้ในการผลิตก็ต่ำประมาณ 6-7 บาทต่อตัวเมื่อคิดจากแรงมือนกดัด ซึ่งหากคิดต้นทุนในลักษณะที่เป็นอุตสาหกรรมแน่นอนว่าจะถูกลงกว่านี้มาก

"จากการศึกษาที่อื่นก็รู้ว่าเหล็กปลอกทั้ง 90 องศา มีพฤติกรรมไม่สู้ดี ไม่เหนียว ฉะนั้นการที่เราใช้กลียัต แล้วทำให้ดีขึ้นก็เป็นประโยชน์ ทำให้พฤติกรรมดีขึ้น ทนต่อแรงโยกได้ดีขึ้น เราไม่ได้หวังว่า

The image is a high-contrast, black and white scan of a document page, likely a ledger or table. It features a grid structure with multiple columns and rows. The text is heavily distorted by noise and artifacts, making it largely illegible. The grid appears to be composed of approximately 10 columns and 15 rows. The right side of the image is dominated by a large, dark, textured area, possibly representing a background or a large block of text that is not clearly defined. The overall quality is poor, with significant visual noise and a high level of contrast.

๑. วัตถุประสงค์ : เพื่อศึกษาเปรียบเทียบผลสัมฤทธิ์ทางการเรียนของนักเรียนชั้นประถมศึกษาปีที่ ๖ ระหว่างการวัดผลสัมฤทธิ์ทางการเรียนโดยใช้แบบทดสอบปรนัย และการวัดผลสัมฤทธิ์ทางการเรียนโดยใช้แบบสัมภาษณ์

๒. ขอบเขตของการวิจัย : ศึกษาเฉพาะนักเรียนชั้นประถมศึกษาปีที่ ๖ โรงเรียนบ้านหนองหญ้าขาว อำเภอเมือง จังหวัดบุรีรัมย์ จำนวน ๓๐ คน และครูผู้สอนจำนวน ๑ คน

๗. ในทะเล	เสียงกรรกรากปล่อยไปลอยไปเป็นไฟ	ทะเลและพ
๘. ฟ้าเป็น	ระลอกน้ำขึ้น นกจับปลาแล้วบิน	นกกินนมดื่

เมื่อถึงเวลา ๑๖.๐๐ น. เรือดำน้ำก็แล่นขึ้นสู่ผิวน้ำ และเรือดำน้ำก็แล่นขึ้นสู่ผิวน้ำ และเรือดำน้ำก็แล่นขึ้นสู่ผิวน้ำ

เมื่อเวลา ๑๖.๐๐ น. พล.ต.ท.วิวัฒน์ พลเดช ผู้บัญชาการตำรวจนครบาล พร้อมด้วย พล.ต.ท.สุวิทย์ วิบุลยสันติกุล ผู้บัญชาการตำรวจภูธรภาค ๑ และ พล.ต.ท.สุวิทย์ วิบุลยสันติกุล ผู้บัญชาการตำรวจภูธรภาค ๑ เดินทางถึงท่าอากาศยานสุวรรณภูมิ เพื่อขึ้นเครื่องบินไปปฏิบัติหน้าที่ที่จังหวัดภูเก็ต

อย่างไรก็ตาม :

โอกาสที่จะเกิดแฉะขึ้น

ไปใช้กับเราต่อม่อละเขา... (๖)

ออกแบบให้แข็งแรงทนใช้

กการวิจัยในครั้งนีศ.ดร.

นเรียมจะไม่รู้สึกอะไรจนกว่าภัยจะมาถึงตัว" นั่นคือประโยคแรกที่เขาอ่านอยู่เสมอระหว่างที่ให้สัมภาษณ์ PEO-
PLE MAGAZINE

ประโยคนี้เองที่ทำให้เขาได้ค้นพบคลิปปิดขาเหล็กปลอกเพื่อช่วยให้เสาคอนกรีตเสริมเหล็กสามารถทนต่อแรงโยกคลอนได้มากขึ้น เพื่อป้องกันอาคารบ้านเรือนไม่ให้เสียหายจากแผ่นดินไหว

จุดประกายเรื่องนี้เมื่อ ดร.นเรียมจากนอกมาหมาดๆ ได้เข้าไปสำรวจและสนใจแผ่นดินไหวที่สร้างความเสียหายให้ประชาชนใน อ.ห้วย จ.เชียงใหม่ เมื่อปี 2519 เขาจะเพิ่งเกิดแผ่นดินไหวอีกครั้งเมื่อปี 2523 เขาได้เข้าศึกษาเรื่องนี้มาตลอด พบว่าในต่างประเทศ ยังมีผู้เชี่ยวชาญ นอกจากนั้นในทางธรณีวิทยายังมีหลายเรื่องที่น่าสนใจ เช่น 500-500 กม. ลมพัดพายุพัดถล่มประเทศไทย ปี 2519 ที่เมืองเชียงใหม่ มีการเสียชีวิต 1 คน

ด้วยความพยายามถึงสองปี (2541-2542) จากทุนวิจัย สกว. หรือสำนักงานกองทุนสนับสนุนการวิจัย สำนัก

ปณิธาน

ลักษณะประสิทธิ์

เมธีวิจัยอาวุโสกับคลิปปักกันแผ่นดินไหว

"ปณิธาน" เกิดมาเพื่อที่จะเป็นนักวิจัยแผ่นดินไหวโดยเฉพาะ ความจงรักภักดีต่อทุกสิ่งที่เป็นแรงผลักดันให้เขาทะลุในแนวทางที่มุ่งหวัง คลิปปิดขาเหล็กปลอกป้องกันแผ่นดินไหว คือผลงานชิ้นเอกที่สร้างชื่อให้เขาเรื่องไกลไปถึงต่างแดน

นายกรัชมณตรี จำนวน 25 ล้านบาท/ปี ทำให้โครงการนี้เป็นเพียงหนึ่งในงานที่ใช้ทุนวิจัยจำนวนมาก ระหว่างที่งานชิ้นแรกได้ดำเนินการทดลองปีละ 1-2 ครั้ง หรือปีละ 1-2 ครั้ง คลิปปิดขาเหล็กปลอก "ปณิธาน" กำลังจะกลายเป็นผลงานชิ้นเอกของเขา หลังจากที่เขาได้แผ่นดินไหว และศึกษาการลดการสั่นไหวของอาคาร เพื่อให้ลดความเสียหาย

ปณิธาน" ในวัย 54 ปีวันนี้ชีวิตได้ประสบความสำเร็จเรื่อยๆ ด้วยความเป็นคนหัวดี มุ่งมั่น และจงรักภักดีต่อทุกสิ่ง เขาได้เหรียญทองอันดับ 1 วิศวกรรมโยธา จากจุฬาลงกรณ์มหาวิทยาลัย แต่กลับได้ทุนมูลนิธิอานันทมหิดล ของคณะวิทยาศาสตร์ไปเรียนต่อปริญญาโทและปริญญาเอก เพราะความหกลดบันต้องการได้ทุนจากที่นี่เพื่อที่จะกลับมาสอนจุฬาลงกรณ์มหาวิทยาลัยเป็นมหาวิทยาลัยเกษตรศาสตร์ให้ทุน

ตรงสายกว่า บังเอิญโชคดีในปีนั้นคณะวิทยาศาสตร์ไม่มีนักศึกษาได้เกียรตินิยมอันดับ 1 เขาจึงได้ไปเรียนที่มหาวิทยาลัยแคลิฟอร์เนีย Berkeley

เมื่อจบกลับมาถึงลอนดอนที่คณะวิศวกรรมศาสตร์ จุฬาลงกรณ์มหาวิทยาลัย มาตลอดจนถึงปัจจุบัน พร้อมกับการทำงานวิจัยเรื่องแผ่นดินไหวตามความสนใจ

เขาเป็นผู้ริเริ่มจัดตั้งหน่วยวิจัยแผ่นดินไหวและการสั่นสะเทือนที่ภาควิชาวิศวกรรมโยธา จุฬาลงกรณ์มหาวิทยาลัยเมื่อปี 2529 ซึ่งต่อมาได้จัดตั้งระบบตรวจวัดอัตราเร่งแผ่นดินไหวในอาคาร รวมทั้งได้ร่วมสร้างอุปกรณ์คอมพิวเตอร์ขนาดใหญ่แห่งแรกในประเทศไทยเพื่อศึกษาวิจัยพฤติกรรมของอาคารภายใต้อิทธิพลของลม เป็นคณะกรรมการแผ่นดินไหวแห่งชาติ และอีกประมาณครึ่งหนึ่งกระดาษไม่สามารถเขียนได้ทั้งหมด

ไม่เพียงแต่ในเมืองไทยเท่านั้น ในต่างประเทศเขายังเคยได้รับเชิญไปร่วมประชุมนานาชาติเกี่ยวกับเรื่องดังกล่าวที่จัดขึ้นโดย UNESCO ฯลฯ

ที่สำคัญผลงานชิ้นล่าสุดของเขา คลิปปิดขาเหล็กปลอกป้องกันแผ่นดินไหว อาจจะนำรายได้เข้าประเทศในเร็ววันนี้ เพราะขณะนี้มันถูกอุตสาหกรรมชาวอเมริกันได้แสดงความสนใจในผลงานชิ้นนี้ และได้ติดต่อขอให้ไปจดสิทธิบัตรที่อเมริกา เพื่อทำการตลาด

บทสัมภาษณ์นี้ได้นำรายได้เข้าประเทศ กำลังจะเกิดขึ้นกับ สกว.แล้ว

๑



ดร.ปณิธาน กับคลิปปิดขาเหล็กปลอกป้องกันแผ่นดินไหว

ที่ ศธ 0209/1134



สำนักงานปลัดกระทรวงสาธารณสุข
ถนนติวานนท์ จ.นนทบุรี 11000

๑๗ ตุลาคม 2543

เรื่อง ขออนุญาตนำผลงานวิจัยมากำหนดเป็นแบบมาตรฐานของกระทรวงสาธารณสุข

เรียน ผู้อำนวยการสำนักงานกองทุนสนับสนุนการวิจัย

ด้วยสำนักงานปลัดกระทรวงสาธารณสุข พิจารณาเห็นว่าผลงานวิจัย เรื่อง MODERATE CONFINEMENT OF RC TIED COLUMNS WITH HOOK-CLIPS FOR MODERATE DUCTILITY ของ ศาสตราจารย์ ดร.ปณิธาน ลักคุณะประสิทธิ์ อาจารย์ประจำคณะวิศวกรรมศาสตร์ ภาควิชาวิศวกรรมโยธา จุฬาลงกรณ์มหาวิทยาลัย ซึ่งเป็นผลงานเกี่ยวกับเหล็กเสริมพิเศษสำเร็จรูปเพื่อรัดเหล็กปลอกกับเหล็กแกนเสา เพื่อเพิ่มกำลังของเสาในการต้านทานแผ่นดินไหว จะเป็นประโยชน์อย่างสูง โดยเพิ่มความแข็งแรงและความปลอดภัยมากยิ่งขึ้น หากสามารถนำมากำหนดเป็นแบบมาตรฐานการเสริมเหล็กเสาคอนกรีตของการก่อสร้างอาคาร ภายใต้การกำกับและดูแลของกระทรวงสาธารณสุข ในบริเวณพื้นที่ 10 จังหวัด ที่กระทรวงมหาดไทยกำหนดให้ออกแบบอาคารต้านแผ่นดินไหว ตามกฎกระทรวง ฉบับที่ 49 (พ.ศ.2540)

สำนักงานปลัดกระทรวงสาธารณสุข จึงประสงค์จะขออนุญาตนำผลงานวิจัยดังกล่าวมากำหนดเป็นแบบมาตรฐาน การเสริมเหล็กเสาคอนกรีตสำหรับการก่อสร้างอาคาร ในบริเวณพื้นที่ 10 จังหวัด ที่กระทรวงมหาดไทยกำหนดให้ออกแบบอาคารต้านแผ่นดินไหว ตามกฎกระทรวง ฉบับที่ 49 (พ.ศ.2540)

จึงเรียนมาเพื่อโปรดอนุญาตต่อไปด้วย จะเป็นพระคุณ

ขอแสดงความนับถือ

(นายปราชญ์ บุณยวงษ์วิโรจน์)

รองปลัดกระทรวง ปฏิบัติราชการแทน
ปลัดกระทรวงสาธารณสุข

กองแบบแผน

โทร. 5901869, 5918565

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EARTHQUAKE HAZARD CENTRE NEWSLETTER

The Commonwealth Network for Earthquake-Resistant Construction in Developing Countries

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Editorial

Connections

How well connected are you? Do you know people who can help you advance professionally or socially? The right connections can be incredibly influential in our lives, particularly when we are experiencing difficulty or hardship.

Connections, albeit of a different nature, are also all-important when it comes to a building forced to survive an earthquake. This is the primary conclusion reached by a team of Italian researchers studying damage to traditional buildings after two damaging earthquakes in Italy in 1977. The buildings, quite typical of most in developing countries, consisted of unreinforced masonry walls, timber floors and clay tiled timber roofs.

G. Sara and his team from the Construction Department of the University of Florence studied the damage patterns of seventy-eight buildings. Their main conclusion highlights the "fundamental importance of connections as viewed in a broad sense". In particular, they commented on the importance of floor to wall connections, and the connections between the outer faces of masonry walls. Of course they observed and commented on other damage, such as that caused by the window and door penetrations that reduce wall in-plane

strength as well. But their final recommendation is that the best way of improving the seismic resistance of existing (and new) buildings is to improve the effectiveness of connections.

These findings should not surprise us. After all, a chain snaps at its weakest link. Certainly when compared to the massiveness and weights of building elements being tied together, such as roofs, floor, gables, and walls, connections are physically small and inherently vulnerable. Overall, seismic performance is largely dominated by the adequacy of small, usually concealed, elements like bolts, steel or stone ties. Performance is dominated by such details or lack of them.

We, as engineers, architects or contractors need to reaffirm the importance of connections. It is quite sobering to reflect on how many factors need consideration in order to design a satisfactory connection. We consider its strength, protection from the elements, constructability, economy and availability. We must also check, in the common case of using steel connectors in conjunction with a weak material like masonry or stone blocks, that the interface connections at each end of the steel are strong enough to transfer forces into the steel. Finally, we need to think through what will happen in the probable event of the connection being overloaded. If at all possible we need to avoid a brittle failure and achieve some ductility. So, even if the connection is damaged, it can still function as a tie between the building elements on either side of it.

Connections are not easy to design or build. They deserve increased attention at all phases of design and construction. During an earthquake they are far more important than our human connections.

Earthquake Hazard Centre

**The Commonwealth Network for Earthquake -
Resistant Construction in Developing Countries**

The Centre is a non-profit organisation supported by the Commonwealth Science Council. It is based at the School of Architecture, Victoria University of Wellington, New Zealand. Director (honorary) and Editor: Andrew Charleson, ME.(Civil)(Dist), MIPENZ; assistance from Ewan MacMaster, BA: David Eyles, Graphic Design.
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significant damage (Fig. 4). This included partial collapse of external walls, collapse of corners, separation of the two walls converging at a corner, and extensive cracking.

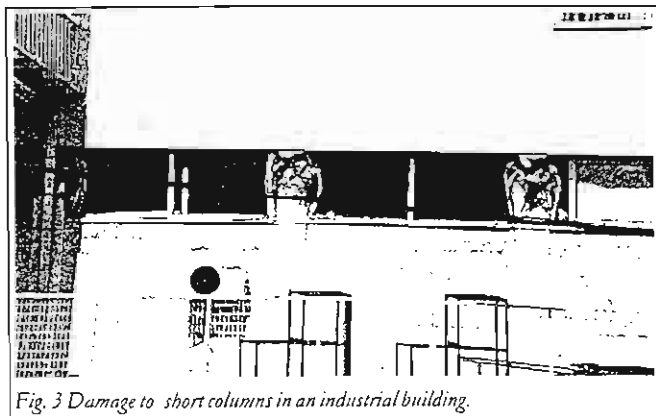


Fig. 3 Damage to short columns in an industrial building.

Brick masonry houses with RC lintel bands or concrete roof slabs, built in recent years, survived with much less damage.

The exceptions were cases in which one or two storeys, supported by an independent RC frame, were added to the top of the masonry house. This is common in Greece in order to extend the building vertically without demolishing the old house. Although the RC structure is statically independent of the masonry building, the slab of first floor is usually connected to the masonry walls. In such "mixed" systems, severe damage occurred to the walls of the masonry house, while the RC structure did not suffer any damage.



Fig. 4 Partial collapse of a stone masonry house.

An Innovative Hook-Clip for Performance Improvement of Reinforced Concrete Tied Columns

Summarised from a paper presented by Panitan Lukkunaprasit at the 12th World Conference on Earthquake Engineering, Auckland, January & February, 2000.

This paper presents a simple-to-place device, called a "hook-clip." It holds the legs of conventional 90° hook ties or cross ties in reinforced concrete columns so as to improve their performance. The purpose of the hook-clip is to prevent premature opening of the legs of 90° hooks. During the study, static compression tests on a number of specimens clearly demonstrated the effectiveness of the clips in restraining the vertical reinforcing bars at the tie positions. Use of the clips resulted in the bars buckling over half the buckling length of the specimens without hook-clips. At a large axial strain of 0.04, columns with hook-clips were able to sustain a loading comparable to that of columns confined with 135° hook ties. Preliminary test results of a medium size column under cyclic loading are also presented.

The deficiency of 90° hook ties in columns has been seen after past earthquakes in bridges and reinforced concrete buildings. Despite their poor performance, 90° hook ties are still used extensively worldwide in low to moderate seismic risk regions because of their ease of placement, when compared with 135° hooks. Ninety-degree hook ties are even more appealing in developing countries where the laying of reinforcing bars is commonly not practised to an acceptable standard of precision, making it extremely difficult to put 135° hook ties in place. In view of the importance of vertical load resisting members, it is extremely desirable to have sufficient inherent ductility (to cope with very rare earthquakes) even in low seismic risk zones. It is also equally important to improve the performance of conventional cross-ties with 90° hooks to ensure their confinement effectiveness in areas of high seismic risk.

To prevent the premature opening of 90° hook ties, a supplementary tie or "hook-clip" is provided which holds the

legs of the hooks and is embedded in the concrete core. The clip resists the opening of the ties after the concrete cover spalls off. Fig. 5 shows the details of the clip proposed for binding 9mm diameter ties or smaller. The hook-clip may be employed to clip the legs of any hoop tie or cross-tie with 90° hooks (see Fig. 6). With the clips prefabricated, the clipping operation can be done easily on-site. No welding is needed.

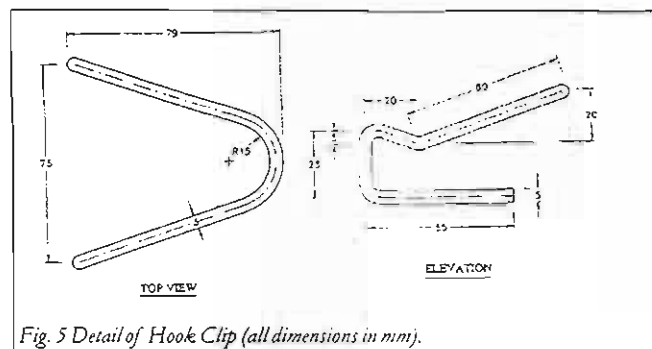


Fig. 5 Detail of Hook Clip (all dimensions in mm).



Fig. 6 Column reinforcing cage with hook clips in place.

Conventional 90° hook-tied columns do not possess sufficient ductility in the event of an earthquake. Hook clips can contribute to earthquake risk reduction in low to moderate seismic risk zones, especially in developing countries where it is difficult to change construction practice. The easy-to-place hook-clips provide an attractive alternative solution, subject to further extensive research work on their cyclic performance.

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รายงานผลการดำเนินงาน

มูลนิธิโทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย ได้รับอนุญาตให้จัดตั้งเป็นทางการ เมื่อวันที่ 2 กุมภาพันธ์ พ.ศ. 2537 โดยมีวัตถุประสงค์เพื่อร่วมส่งเสริมความก้าวหน้าทางด้านวิทยาศาสตร์และเทคโนโลยีในประเทศไทย

เพื่อบรรลุประสงค์ดังกล่าว มูลนิธิฯ ได้ดำเนินกิจกรรมเพื่อการพัฒนาและส่งเสริมความก้าวหน้าทางด้านวิทยาศาสตร์และเทคโนโลยีในประเทศไทย โดยจัดให้มีกิจกรรมสามประเภทด้วยกัน

- ประเภทแรก คือ การจัดให้มีรางวัลวิทยาศาสตร์และเทคโนโลยี สำหรับบุคคลหรือสถาบัน ที่มีผลงานดีเด่นด้านวิทยาศาสตร์และเทคโนโลยี
- ประเภทที่สอง คือ การให้เงินทุนช่วยเหลือทางด้านวิจัยวิทยาศาสตร์และเทคโนโลยี เพื่อสนับสนุน อาจารย์ นักศึกษา นักวิจัยที่กำลังค้นคว้าหรือมีโครงการค้นคว้าวิจัยที่เป็นรากฐานที่จะอำนวยความสะดวกให้แก่วงการวิทยาศาสตร์และเทคโนโลยีในประเทศไทย
- ประเภทที่สาม คือ รางวัลการศึกษาวิทยาศาสตร์ โดยจะมอบให้แก่บุคลากรผู้รับผิดชอบทางการเรียนการสอนวิชาวิทยาศาสตร์ของโรงเรียนในระดับมัธยมศึกษาตอนต้นและตอนปลาย ที่มีผลงานดีเด่นในการสร้างสรรค์และริเริ่มทางการศึกษาวิทยาศาสตร์ เพื่อนำไปพัฒนาและเพิ่มพูนความสนใจของนักเรียนต่อการเรียนวิชาวิทยาศาสตร์ นอกจากนี้ ในปีพุทธศักราช 2539 มูลนิธิฯ ยังได้ให้การสนับสนุนแก่ห้องปฏิบัติการวิทยาศาสตร์ของโรงเรียนที่ได้รับรางวัลการศึกษาวิทยาศาสตร์อีกด้วย

มูลนิธิโทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย ได้รับเงินกองทุนประเดิมจาก Toray Industries, Inc., Japan โดยใช้ดอกผลจากกองทุนนี้ นอกจากนี้ยังได้รับเงินบริจาคจาก Toray Science Foundation, Japan และกลุ่มบริษัทโทเรในประเทศไทย 5 บริษัท

รางวัลวิทยาศาสตร์และเทคโนโลยี

ในปีพุทธศักราช 2544 ซึ่งเป็นปีที่แปดของการดำเนินกิจกรรมนี้ ในด้านบุคคลที่มีผลงานทางด้านวิทยาศาสตร์และเทคโนโลยีที่มีคุณภาพเป็นเลิศ ซึ่งได้รับการตีพิมพ์ในวารสารวิชาการที่มีการตรวจสอบคุณภาพอย่างเคร่งครัด ตลอดจนเป็นผลงานที่มีคุณค่าต่อสังคมในด้านการสร้างความก้าวหน้าทางวิชาการและในด้านศักยภาพของการนำไปประยุกต์ใช้ คณะกรรมการสาขารางวัลวิทยาศาสตร์และเทคโนโลยี มูลนิธิโทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย ได้พิจารณาผลงานของบุคคล/สถาบัน ที่ได้รับการเสนอชื่อและได้เสนอคณะกรรมการบริหารมูลนิธิฯ ซึ่งได้มีมติเป็นเอกฉันท์ยกย่องให้ ศาสตราจารย์ ดร. ปณิธาน ลักคุณะประสิทธิ์ หัวหน้าหน่วยวิจัยแผ่นดินไหวและการสั่นสะเทือน คณะวิศวกรรมศาสตร์ จุฬาลงกรณ์มหาวิทยาลัย เป็นผู้ได้รับรางวัลวิทยาศาสตร์และเทคโนโลยี ในฐานะที่เป็นนักวิทยาศาสตร์ผู้มีผลงานวิจัยตีพิมพ์ต่อเนื่องในวารสารระดับนานาชาติ นับเป็นผู้บุกเบิกและเผยแพร่ความรู้ทางด้านวิทยาศาสตร์ เทคโนโลยีและวิศวกรรมศาสตร์ให้เป็นที่รู้จักกันอย่างแพร่หลายรวมทั้งการสร้างบุคลากรรุ่นใหม่ทางด้านวิทยาศาสตร์และเทคโนโลยีของประเทศ ตลอดจนการนำผลงานวิจัยไปประยุกต์ใช้เพื่อความปลอดภัย คุณภาพชีวิตของประชาชนคนไทยและด้านเศรษฐกิจของประเทศ

ศาสตราจารย์ ดร. ปณิธาน ลักคุณะประสิทธิ์ เป็นผู้หนึ่งที่ริเริ่มบุกเบิกค้นคว้าวิจัยเพื่อนำไปสู่การใช้ประโยชน์ในการออกแบบอาคารให้ต้านภัยจากแผ่นดินไหวและवादภัยได้โดยประหยัดและปลอดภัย เพื่อการพัฒนาที่ยั่งยืน ผลงานวิจัยเกี่ยวกับความเร็วลมและแรงลมได้ถูกใช้เป็นพื้นฐานในการกำหนดความเร็วลมในมาตรฐาน “การคำนวณแรงลมสำหรับการออกแบบ



อาคาร” ของสมาคมวิศวกรรมสถานแห่งประเทศไทยฯ ทำให้ได้มาตรฐานทัดเทียมประเทศที่พัฒนาแล้ว

ด้านผลของแผ่นดินไหว ได้ทำการศึกษาวิจัยเกี่ยวกับพฤติกรรมเชิงวัฏจักรของเสาซึ่งเสริมด้วยเหล็กเสริมโอบรัศระดับปานกลาง โดยพิจารณารวมถึงระดับแรงอัดตามแนวแกนและรูปแบบของเหล็กปลอกเดี่ยว ผลงานวิจัยนี้มีคุณค่าในการช่วยเสริมข้อมูลและองค์ความรู้ให้สมบูรณ์ในย่านความเหนียวปานกลาง ซึ่งมีประโยชน์โดยตรงกับประเทศไทย รวมทั้งประเทศหรือถิ่นที่มีแผ่นดินไหวขนาดปานกลาง นอกจากนี้ได้พัฒนาสิ่งประดิษฐ์คือ คลิปปิดขาของเหล็กปลอก (hook-clip) เพื่อปรับปรุงความเหนียวของเสาคอนกรีตเสริมเหล็กอย่างมีประสิทธิภาพและสามารถประยุกต์ใช้ได้จริง อีกทั้งได้ศึกษาผลของตัวหน่วงปรับมวล (tuned mass damper, TMD) ในการลดการสั่นไหวของอาคารจากผลของแผ่นดินไหวระยะไกลที่อาจมีผลมายังกรุงเทพฯ โดยที่งานวิจัยนี้มีประโยชน์สำหรับการประยุกต์ใช้ในการออกแบบบนพื้นฐานสมรรถนะ (performance-based design) ของโครงสร้าง

สำหรับหน่วยงานที่ได้รับการคัดเลือกให้รับรางวัลวิทยาศาสตร์และเทคโนโลยี คือ ห้องปฏิบัติการวิจัยเคมีซูพราโมเลคิวลาร์ (Supramolecular Chemistry Research Laboratory) ภาควิชาเคมี คณะวิทยาศาสตร์ จุฬาลงกรณ์มหาวิทยาลัย โดยผลงานของห้องปฏิบัติการวิจัยเคมีซูพราโมเลคิวลาร์ สามารถแบ่งออกเป็น 2 ด้านคือ การหาคำตอบใหม่และการประยุกต์ใช้ความรู้ด้านเคมีซูพราโมเลคิวลาร์ ตลอดจนพัฒนาวิทยาการด้านนี้ให้สามารถทัดเทียมกับนานาชาติประเทศ อาทิ การตรวจจับหรือตรวจวัดสปีชีส์ต่างๆ ที่มีความสำคัญและมีผลกระทบด้านสิ่งแวดล้อมและชีวภาพ การวิจัยเกี่ยวกับเรื่องโมเลคิวลาร์สวิตช์ (molecular switch) ซึ่งสามารถนำไปประยุกต์ใช้เป็นเครื่องมือระดับโมเลกุลได้ การสร้างเซ็นเซอร์ที่สามารถตรวจวัดสารต่างๆ ได้ด้วยตาเปล่าหรือด้วยเทคนิคที่มีราคาถูก การสังเคราะห์พอลิเมอร์ที่มีโลหะเป็นส่วนประกอบเพื่อนำไปใช้ทำวัสดุชนิดใหม่ อนึ่ง การดำเนินงานของห้องปฏิบัติการฯ นับเป็นการวิจัยทางด้านนาโนเคมีที่จะมีบทบาทสำคัญอย่างยิ่งต่อความเจริญก้าวหน้าทางวิทยาศาสตร์และเทคโนโลยีของประเทศไทยในศตวรรษที่ 21 ซึ่งเป็นยุคที่งานวิจัยทางด้านนาโนเทคโนโลยีจะก้าวเข้ามามีบทบาทมากยิ่งขึ้น

ผลงานอีกด้านหนึ่งของห้องปฏิบัติการฯ คือ การสร้างนักคิดและนักวิจัยรุ่นใหม่ๆ ออกสู่สังคม โดยการถ่ายทอดความรู้ แนวคิดรวบยอด ตลอดจนทักษะในการทำวิจัยด้านเคมีซูพราโมเลคิวลาร์ให้แก่แก่นักวิจัยรุ่นใหม่ๆ กล่าวคือ แม้ว่าห้องปฏิบัติการฯ จะเป็นเพียงหน่วยงานเล็กๆ ที่มีผู้ร่วมงานเป็นเพียงคณะอาจารย์ โดยไม่มีนักเทคนิคหรือผู้ช่วยวิจัยอื่นๆ แต่ห้องปฏิบัติการฯ สามารถดำเนินการวิจัยและสร้างนักวิจัยรุ่นใหม่ๆ ได้อย่างต่อเนื่อง ปัจจุบันมีนิสิตระดับปริญญาตรี โท และเอกที่เข้าร่วมดำเนินงานกับห้องปฏิบัติการฯ รวม 20 คน ที่ผ่านมาห้องปฏิบัติการฯ สามารถผลิตบัณฑิตในระดับปริญญาตรีและโทที่ทำวิจัยกับห้องปฏิบัติการฯ และสำเร็จการศึกษาแล้วรวม 40 คน

ทุนช่วยเหลือทางด้านวิจัยวิทยาศาสตร์และเทคโนโลยี

นอกจากรางวัลสาขาวิทยาศาสตร์และเทคโนโลยีแล้ว มูลนิธิฯ ยังได้ให้ทุนช่วยเหลือทางด้านวิจัยวิทยาศาสตร์และเทคโนโลยี โดยที่หัวข้องานวิจัยจะต้องเป็นประโยชน์ต่อส่วนรวมและต่อการพัฒนาองค์ความรู้ทางวิทยาศาสตร์และเทคโนโลยีของประเทศ โดยในปีพุทธศักราช 2544 นี้ได้มอบทุนช่วยเหลือทางด้านวิจัยวิทยาศาสตร์และเทคโนโลยี รวมทั้งสิ้น 15 ทุนวิจัย ดังนี้



สาขาเกษตรศาสตร์ มีจำนวน 5 โครงการ

1. ชื่อโครงการ การปรับปรุงพันธุ์ให้มีลักษณะฝักไม่แตกและฝักต้านทานการแตก
 ชื่อนักวิจัย รองศาสตราจารย์वासนา วงษ์ใหญ่
 หน่วยงาน ภาควิชาพืชไร่นา มหาวิทยาลัยเกษตรศาสตร์
2. ชื่อโครงการ ศักยภาพในการใช้ Rhizobacteria ในธรรมชาติสำหรับการส่งเสริม
 การเจริญเติบโตและการเพิ่มผลผลิตของพืช
 ชื่อนักวิจัย ผู้ช่วยศาสตราจารย์กัญชลี เจตยานนท์
 หน่วยงาน ภาควิชาวิทยาศาสตร์การเกษตร มหาวิทยาลัยนเรศวร
3. ชื่อโครงการ กระบวนการผลิตข้าวเหนียวหุงสุกแช่เยือกแข็ง
 ชื่อนักวิจัย รองศาสตราจารย์อรอนงค์ นัยวิกุล
 หน่วยงาน คณะอุตสาหกรรมเกษตร มหาวิทยาลัยเกษตรศาสตร์
4. ชื่อโครงการ การพัฒนาพันธุ์ถั่วเหลืองฝักสดลูกผสมและเทคนิคการผลิตเมล็ดพันธุ์
 ถั่วเหลืองฝักสดในระดับเกษตรกร
 ชื่อนักวิจัย รองศาสตราจารย์ ดร. วิทยา บัวเจริญ
 หน่วยงาน คณะเทคโนโลยีการเกษตร
 สถาบันเทคโนโลยีพระจอมเกล้าเจ้าคุณทหารลาดกระบัง
5. ชื่อโครงการ ศักยภาพของการใช้เชื้อผสมของ *Bacillus* spp. ที่แยกได้จากลำไส้ของ
 กุ้งก้ามกรามเป็นโปรไบโอติกในอาหารเลี้ยงกุ้ง และการศึกษาภูมิคุ้มกันโรค
 ในกุ้งหลังจากได้รับโปรไบโอติก
 ชื่อนักวิจัย นายวิชัย ลีลาวัชรมาศ
 หน่วยงาน คณะเทคโนโลยี มหาวิทยาลัยขอนแก่น

สาขาเคมี มีจำนวน 4 โครงการ

6. ชื่อโครงการ การศึกษาผลของการเชื่อมโยงทางเคมีในวัฏภาคของยางโดยใช้กัมมันต์
 ที่มีต่อสมบัติความทนน้ำมันและความร้อนของพอลิเมอร์ผสมระหว่าง
 ยางธรรมชาติกับคลอรีนเตฟพอลิเอธิลีน
 ชื่อนักวิจัย ผู้ช่วยศาสตราจารย์ชาคริต ลีริสิงห์
 หน่วยงาน คณะวิทยาศาสตร์ มหาวิทยาลัยมหิดล
7. ชื่อโครงการ การเตรียมยางสีกมโปรตีนต่ำเพื่อใช้เป็นวัตถุดิบสำหรับกาวชนิดพิเศษ
 ที่ไม่ทำให้เกิดภูมิแพ้
 ชื่อนักวิจัย ผู้ช่วยศาสตราจารย์จิตต์ลัดดา ศักดาภิพาณิชย์
 หน่วยงาน ภาควิชาเคมี คณะวิทยาศาสตร์ มหาวิทยาลัยมหิดล
8. ชื่อโครงการ การแยกและศึกษาหน้าที่ของยีนที่เกี่ยวข้องกับชีวสังเคราะห์
 สารแอนโทไซยานินจากกระเพรา
 ชื่อนักวิจัย รองศาสตราจารย์ ดร. ดุชนิ ธนะบริพัฒน์
 หน่วยงาน คณะวิทยาศาสตร์ สถาบันเทคโนโลยีพระจอมเกล้าเจ้าคุณทหารลาดกระบัง
9. ชื่อโครงการ การศึกษาปฏิกิริยาโฟทอกิลเลชั่นแบบเฉพาะของน้ำตาล เพื่อนำไปสู่
 การสังเคราะห์สารที่มีฤทธิ์ทางชีวภาพ
 ชื่อนักวิจัย นางสาวรุ่งนภา แซ่เอ็ง
 หน่วยงาน ภาควิชาเคมี มหาวิทยาลัยบูรพา



สาขาวิศวกรรมศาสตร์ มีจำนวน 5 โครงการ

10. ชื่อโครงการ การปรับปรุงคุณภาพผิวและการควบคุมขนาดของผลิตภัณฑ์พีวีซีที่ได้จากกระบวนการอัดรีดโดยระบบหัวขึ้นรูปสนามแม่เหล็ก
 ชื่อนักวิจัย ดร. ณรงค์ฤทธิ์ สมบัติสมภพ
 หน่วยงาน คณะพลังงานและวัสดุ มหาวิทยาลัยเทคโนโลยีพระจอมเกล้าธนบุรี
11. ชื่อโครงการ การศึกษาความเป็นไปได้ของการเกิดสภาวะ Liquefaction ของชั้นทรายในจังหวัดทางภาคเหนือของประเทศไทย
 ชื่อนักวิจัย ดร. สุพจน์ เดชวรสินสกุล
 หน่วยงาน คณะวิศวกรรมศาสตร์ จุฬาลงกรณ์มหาวิทยาลัย
12. ชื่อโครงการ อเสถียรภาพของพื้นที่เชิงเขาจากการเกิดฝนตกอย่างหนักต่อเนื่องในบริเวณรับน้ำของเทือกเขา
 ชื่อนักวิจัย ผู้ช่วยศาสตราจารย์กฤษณะเดช เจริญสุธาสิน
 หน่วยงาน ภาควิชาฟิสิกส์และวิทยาศาสตร์เชิงคำนวณ มหาวิทยาลัยวลัยลักษณ์
13. ชื่อโครงการ การวิเคราะห์เศษโลหะจากปลั๊กแม่เหล็ก เพื่อประเมินสภาพเครื่องจักรกลอุตสาหกรรม
 ชื่อนักวิจัย ดร. สุรพล ราษฎร์นุ้ย
 หน่วยงาน คณะวิศวกรรมศาสตร์ สถาบันเทคโนโลยีพระจอมเกล้าพระนครเหนือ
14. ชื่อโครงการ กลไกโฟนอน: เหตุแห่งการสั่นของแขนกลอ่อนตัว
 ชื่อนักวิจัย ผู้ช่วยศาสตราจารย์ปิยะเดช สุรักษา
 หน่วยงาน ภาควิชาวิศวกรรมสารสนเทศ สถาบันเทคโนโลยีพระจอมเกล้าเจ้าคุณทหารลาดกระบัง

สาขาฟิสิกส์ มีจำนวน 1 โครงการ

15. ชื่อโครงการ การออกแบบและสร้างซอฟต์แวร์สำหรับสร้างโมเดลและให้กำเนิดโปรแกรมควบคุมการทำงานโดยอัตโนมัติสำหรับระบบ Real-Time ขนาดเล็ก
 ชื่อนักวิจัย ดร. วัฒนพงศ์ เกิดทองมี
 หน่วยงาน ภาควิชาฟิสิกส์ มหาวิทยาลัยวลัยลักษณ์

รางวัลการศึกษาวิทยาศาสตร์

สำหรับผลการตัดสินรางวัลการศึกษาวิทยาศาสตร์นั้น คณะกรรมการการศึกษาวิทยาศาสตร์ ได้คัดเลือกให้มีผู้ได้รับรางวัลในระดับมัธยมศึกษาตอนต้นและมัธยมศึกษาตอนปลาย ระดับละ 3 รางวัล โดยในปีนี้มีผู้ได้รับรางวัลที่ 1 ในระดับมัธยมศึกษาตอนต้น รายละเอียดมีดังนี้

ระดับมัธยมศึกษาตอนต้น

รางวัลที่ 2 เงินรางวัล 60,000 บาท ได้แก่
 อาจารย์ประเสริฐ นาคีสินธุ์
 โรงเรียนธรรมศาสตร์-จุฬา 2 อำเภอสวนผึ้ง จังหวัดราชบุรี
 ชื่อผลงาน : เครื่องอบไอน้ำในโรงเรือนเห็ดฟางปลอดมลพิษ

รางวัลที่ 2 เงินรางวัล 60,000 บาท ได้แก่
 อาจารย์ภูมิ มุ่งตอกิจ



โรงเรียนพุทธรังสิทิบูล อำเภอบ้านโพธิ์ จังหวัดฉะเชิงเทรา

ชื่อผลงาน : แม่เหล็กไฟฟ้าและแรงขับเคลื่อนแม่เหล็ก

รางวัลที่ 3 เงินรางวัล 40,000 บาท ได้แก่
อาจารย์ชาญชิต พงศ์วิทยานนท์
โรงเรียนน้ำพองศึกษา อำเภอน้ำพอง จังหวัดขอนแก่น
ชื่อผลงาน : ชุดควบคุมเวลาระบบน้ำหยด

ระดับมัธยมศึกษาตอนปลาย

รางวัลที่ 2 เงินรางวัล 80,000 บาท ได้แก่
อาจารย์ปิ่น ช่างทอง
โรงเรียนมัธยมสิริวัณวรี 3 ฉะเชิงเทรา อำเภอบ้านโพธิ์ จังหวัดฉะเชิงเทรา
ชื่อผลงาน : การศึกษาเห็บในเขตรักษาพันธุ์สัตว์ป่าเขาอ่างฤๅไน

รางวัลที่ 2 เงินรางวัล 80,000 บาท ได้แก่
อาจารย์ภาณี หนูทอง
โรงเรียนพัทลุง อำเภอเมือง จังหวัดพัทลุง
ชื่อผลงาน : การเพาะเลี้ยงด้วงงวงมะพร้าว

รางวัลที่ 3 เงินรางวัล 60,000 บาท ได้แก่
อาจารย์วิชาญ ภูผาคูณ
โรงเรียนบัวขาว อำเภอกุฉินารายณ์ จังหวัดกาฬสินธุ์
ชื่อผลงาน : ชุดสาธิตการทดลองความเค้นตามยาวและความเครียดตามยาว

โรงเรียนที่ได้รับเงินสนับสนุนห้องปฏิบัติการทางวิทยาศาสตร์ โรงเรียนละ 20,000 บาท ได้แก่

1. โรงเรียนธรรมศาสตร์-จุฬา 2 อำเภอสวนผึ้ง จังหวัดราชบุรี
2. โรงเรียนพุทธรังสิทิบูล อำเภอบ้านโพธิ์ จังหวัดฉะเชิงเทรา
3. โรงเรียนน้ำพองศึกษา อำเภอน้ำพอง จังหวัดขอนแก่น
4. โรงเรียนมัธยมสิริวัณวรี 3 ฉะเชิงเทรา อำเภอบ้านโพธิ์ จังหวัดฉะเชิงเทรา
5. โรงเรียนพัทลุง อำเภอเมือง จังหวัดพัทลุง
6. โรงเรียนบัวขาว อำเภอกุฉินารายณ์ จังหวัดกาฬสินธุ์

มูลนิธิโทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย ขอขอบคุณมูลนิธิโทเร เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย กลุ่มบริษัทโทเรในประเทศไทย และคณะกรรมการชุดต่างๆ ที่ได้ดำเนินการคัดเลือกผู้ได้รับรางวัล และหวังว่าความพยายามของมูลนิธิฯ ในการดำเนินกิจกรรมดังที่ได้กล่าวมานี้ จะเป็นส่วนสำคัญส่วนหนึ่งของสังคมที่จะช่วยจรรโลงวงการวิทยาศาสตร์ของประเทศไทยให้เจริญรุดหน้าสืบต่อไป



ศาสตราจารย์ ดร. สิปปนนท์ เกตุทัต
ประธานมูลนิธิโทเร
เพื่อการส่งเสริมวิทยาศาสตร์ ประเทศไทย

เทียบ รรหอที่๑

าชน

หายไป จนกระทั่ง
ได้ตอบกลับมาใน
และนำสู่ที่ประชุม

ยสิทธิ์ไปถึง 1 ปี
งเท่าเดิม เราไม่ได้
ก แต่มันเป็นสิทธิ์
อยู่แล้ว หากไม่
หมาย เรื่องนี้เป็น
กระทรวงแรงงาน
ถึง เพราะนายเดช
กการกระทรวงแรง
นไม่เคยเข้าประชุม
กเป็นรัฐมนตรี.เพียง
าประชุม" นาย

ก่อนหน้านี้ ลูกจ้าง
1.2 หมื่นคน ล่าสุด
งานอยู่ประมาณ
ยเงินย้อนหลังเพิ่ม
165 บาท จะเท่ากับ
3 บาทต่อวัน คิด
จ่ายเพิ่มทั้งสิ้น
ท ซึ่งหากรายได้ที่
มารวมกลับมาขอ

คลิปต้านแผ่นดินไหวคว่ำรางวัลโทะเร

อาจารย์จุฬาฯ เจ้าของ "คลิปยัดขาของ
เหล็กปลอก" คว่ำรางวัลนักวิทยุดีเด่นปี
2545 เผยจดสิทธิบัตรแล้ว ส่วนผลงาน
วิทยาศาสตร์เด่นในรอบปี คือการศึกษา
ยีนกะเพราแดงเพื่อตัดแปลงพันธุกรรม
ให้สีกับดอกกุหลาบ

มูลนิธิโทะเรเพื่อการส่งเสริม
วิทยาศาสตร์ ประเทศไทย มอบรางวัล
นักวิทยาศาสตร์และเทคโนโลยีประจำปี
2545 โดยมี ศ.ดร.พณฯ พล.เอก.ประม
ติณสุลานนท์ ประธานองคมนตรีและ
รัฐบุรุษ เป็นประธาน ได้มอบรางวัลนัก
วิทยาศาสตร์ดีเด่นให้แก่ ดร.ปณิธาน
ลักคุณะประสิทธิ์ หัวหน้าวิจัยแผ่นดิน
ไหวและสั่นสะเทือน คณะวิศวกรรม
ศาสตร์ จุฬาลงกรณ์มหาวิทยาลัย

ในฐานะที่เป็นผู้เริ่มต้นคว้า
วิศวกรรมแผ่นดินไหวและแรงสั่น
ในประเทศไทย ที่สามารถนำไปใช้
ประโยชน์ได้จริง ผลงานที่โดดเด่น คือ
การออกแบบอาคารต้านภัยแผ่นดิน
ไหวและवादภัยได้โดยประหยัดและ
ปลอดภัย โดยสามารถพัฒนา "คลิปยัด
ขาของเหล็กปลอก" เพื่อปรับปรุงความ
เหนียวของเสาคอนกรีตเสริมเหล็กลง
มีประสิทธิผล และสามารถประยุกต์
ใช้ได้จริง

นอกจากนี้มูลนิธิโทะเร ได้มอบ
รางวัลวิทยาศาสตร์ประเภทหน่วยงาน
ให้แก่ห้องปฏิบัติการวิจัยเคมีซูปราโมเล
คิวลาร์ ภาควิชาเคมี คณะวิทยาศาสตร์
จุฬาลงกรณ์มหาวิทยาลัย ผลงานที่
สำคัญของห้องปฏิบัติการนี้คือ การหา
องค์ความรู้ใหม่และการประยุกต์ใช้
ความรู้ด้านเคมีซูปราโมเลคิวลาร์ ซึ่ง
เป็นวิทยาการโดดเด่นทางด้านนาโน
เคมีที่มีบทบาทสำคัญในการพัฒนา
วิทยาศาสตร์ของประเทศไทยใน
ศตวรรษที่ 21

ดร.ปณิธาน กล่าวว่า การพัฒนา
"คลิปยัดขาของเหล็กปลอก" เพื่อปรับ
ปรุงความเหนียวของเสาคอนกรีตเสริม
เหล็ก จะสามารถป้องกันการสั่นสะเทือน
ของแผ่นดินไหวได้อย่างมีประสิทธิภาพ
ซึ่งที่ผ่านมา การออกแบบอาคารใน
ประเทศไทยยังไม่ได้มีการป้องกันแผ่นดิน
ไหว ซึ่งหลังจากที่มีกฎหมายให้มีการ
ออกแบบอาคารเพื่อป้องกันแผ่นดินไหว
ใน 9 จังหวัดภาคเหนือ และกาญจนบุรี
ผลงานชิ้นนี้จะมีประโยชน์ต่อการก่อสร้าง
มากขึ้น

ดร.ปณิธาน กล่าวว่า คลิปยัด
เหล็กอยู่ระหว่างการยื่นขอจดสิทธิบัตร
2 ปีแล้ว แต่ยังไม่สามารถพัฒนาสู่เชิง

พาณิชย์ จึงไม่มีอาคารไหนนำไปใช้
ประโยชน์ แต่อย่างไรก็ตาม กระทรวง
สาธารณสุขได้ติดต่อมายังสำนักงานกอง
ทุนสนับสนุนการวิจัย (สกว.) ที่เป็น
ผู้สนับสนุนการวิจัย เพื่อนำเอาสิ่ง
ประดิษฐ์ดังกล่าวไปใช้ประโยชน์ โดย
จะนำไปก่อสร้างอาคารของกระทรวง
สาธารณสุขในพื้นที่ 10 จังหวัด ในภาค
เหนือ 9 จังหวัดและกาญจนบุรี คิดว่าใน
อนาคตผลงานชิ้นนี้จะสามารถนำไป
พัฒนาในเชิงพาณิชย์ได้

ดร.กนกพร สมพรโพลิน คณะ
วิทยาศาสตร์ สถาบันเทคโนโลยีพระ
จอมเกล้าเจ้าคุณทหารลาดกระบัง
(สจล.) กล่าวถึงงานวิจัยการแยกและ
ศึกษาหน้าที่ของยีนที่เกี่ยวข้องกับ
ชีวสังเคราะห์สารแอนโทไซยานินจากกะ
เพรา ว่า เนื่องจากในใบกะเพราแดง
สามารถผลิตสารกลุ่มแอนโทไซยานินใน
ปริมาณสูง โดยสารกลุ่มนี้เป็นกลุ่ม
ฟลาโวนอยด์ ที่มีคุณสมบัติเป็นสาร
เสริมสุขภาพ โดยแอนโทไซยานินแสดง
ลักษณะการทำงานหลายอย่างในเซลล์
สัตว์ เช่น แอนติออกซิแดนท์และป้องกัน
การเกิดมะเร็ง ซึ่งการตัดต่อยีนกะเพรา
แดง จะนำไปประยุกต์ใช้ในการตัดต่อยีน
ให้สีกับดอกกุหลาบได้

ไวรัสเอดส์กลายพันธุ์วัคซีนเท่านั้นหยุดได้

เผยปัจจุบันมีประชากรทั่วโลกประมาณ
36 ล้านคนที่ติดเชื้อไวรัสเอชไอวี และในทุก ๆ
วันมีคนติดเชื้อเพิ่ม วันละ 15,000 คน ล่าสุด
บริษัทยาต่างแข่งขันกันผลิตยาป้องกันการ
ติดเชื้อเอดส์ ล่าสุดบ.สมิธ ไคลน์ ตามมาด้วย
บริษัทเมอร์ค จะสำเร็จในอีก 10 ปีข้างหน้า
แต่เนื่องจากไวรัสดังกล่าวกลายพันธุ์อย่าง
รวดเร็ว ทำให้ตัวยา ยาที่ใช้ในปัจจุบันเริ่มใช้ไม่ได้
แล้ว ความหวังเดียวคือการผลิตวัคซีนที่มีฤทธิ์
ป้องกัน

บริษัทแกล็กโซ สมิธ ไคลน์ ประเทศ
อังกฤษ จะเริ่มการทดลองยาป้องกันการติด
เชื้อไวรัสเอชไอวี ซึ่งเป็นสาเหตุการเกิดโรค
เอดส์ การทดลองจะมีขึ้นที่สหรัฐ ในศูนย์การ
แพทย์ 11 แห่งทั่วประเทศ ยานานนี้ได้รับการ
พัฒนาขึ้น ซึ่งประกอบด้วยโปรตีนสำคัญ 2 ชนิด
ที่พบอยู่ในไวรัสเอชไอวี

ให้วัคซีนมีประสิทธิภาพมากขึ้น วัคซีนดังกล่าว
จะมีประสิทธิภาพก็ต่อเมื่อให้กับคนที่ยัง
ไม่เคยติดเชื้อไวรัสเอชไอวีมาก่อน

ซึ่งจากการทดลองก่อนหน้านี้กับลิง
ปรากฏว่า วัคซีนช่วยปกป้องลิงจากไวรัส
เอชไอวีซึ่งมีลักษณะคล้ายๆ กับไวรัสเอชไอ
วีที่พบในมนุษย์ แต่เมื่อนำมาทดลองในมนุษย์
จะเหมือนกับที่ทดลองจากลิงหรือเป่ายังไม่
สามารถตอบได้

แต่จากการทดลองในลิง พบว่า ยาตัวนี้
ของบริษัทแกล็กโซ ยังคงรักษาระดับของ
เซลล์ที่มีชื่อว่า "ซีดีโฟร์" ซึ่งเป็นเซลล์ภูมิคุ้มกัน
ที่ช่วยร่างกายต่อต้านเชื้อแบคทีเรียและไวรัส
การทดลองในขั้นแรกเป็นการทดสอบความ
ปลอดภัยโดยที่ยังไม่ต้องคำนึงถึงประสิทธิภาพ
จึงมุ่งไปที่การทดลองกลุ่มคนที่มีความเสี่ยงต่ำ
ในการติดเชื้อ

ทำให้กับกลุ่มประชากรที่มีความเสี่ยงสูง ยา
ของบริษัทสมิธไคลน์มีคู่แข่งสำคัญ คือ ยา
ของบริษัทเมอร์ค ซึ่งอยู่ในขั้นแรกของการ
ทดลอง คาดว่าจะประสบความสำเร็จภายใน
10 ปีข้างหน้า ในปัจจุบันมีประชากรทั่วโลก
รวมประมาณ 36 ล้านคนที่ติดเชื้อไวรัสเอช
ไอวี และในทุก ๆ วันมีคนติดเชื้อเพิ่ม วันละ
15,000 คน

อย่างไรก็ตาม ถึงแม้ว่ามีการผลิตย
ต่อต้านไวรัสเอชไอวี ในปัจจุบันซึ่งใช้หลาย
ขนานรวมกันสามารถควบคุมเชื้อไวรัสและ
หยุดยั้งความอันตรายของมันได้ แต่ก็พบว่า
ไวรัสเอชไอวีเกิดการกลายพันธุ์อย่างรวดเร็ว
ทำให้เกิดการดื้อยา

ซึ่งยาที่ใช้อยู่ปัจจุบันเริ่มใช้ไม่ได้กั
คนใช้จำนวนมากขึ้น ความหวังที่แท้จริงใ
ปัจจุบันจึงอยู่ที่การผลิตวัคซีนที่มีฤทธิ์เ