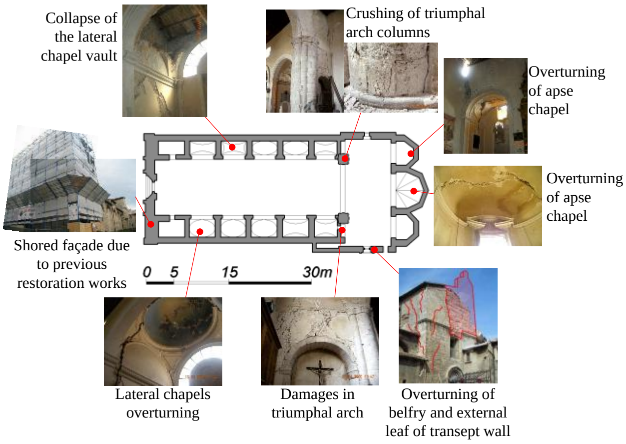

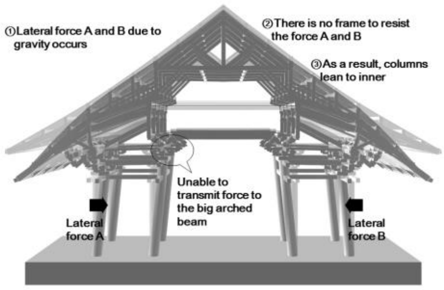

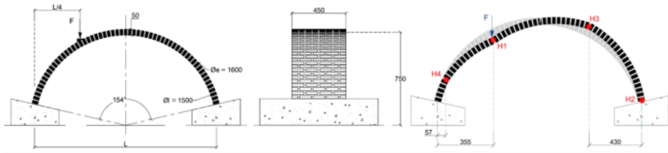

International Workshop by Young Researchers for

"Application of Structural Engineering and Structural Health Monitoring to Historic Buildings"

December 19, 2014
DPRI, Kyoto University

Sponsor: Disaster Prevention Research Institute, Kyoto University

DPRI-KU

京都大学
KYOTO UNIVERSITY

Disaster Prevention Research Institute, Kyoto University
Gokasho, Uji
Kyoto 611-0011, JAPAN

December 19, 2014

Dear Participants,

I would like to extend my personal welcome to DPRI, Kyoto University and to the International Workshop by Young Researchers for “Application of Structural Engineering and Structural Health Monitoring to Historic Buildings”. I am very excited for intellectual challenges we will explore together. In the world, many historic buildings suffer damage from natural and/or man-made disasters every year. Restoration and condition evaluation of these structures against gravity and natural hazards are our duty as historic buildings shall be securely inherited to our future generations.

In Japan, most of the buildings categorized as “historic” are traditional wooden buildings. Meanwhile, not a few masonry buildings built in the modern period of the Meiji (1868-1912), Taisho (1912-1926) and early Showa (1926-1989) eras exists as well. In many places of the world, traditional masonry buildings are more common type of historic buildings. The construction technique and structural concept of historic buildings may be different from the today’s construction practice and often disqualified by the current building codes and specifications. Thus, this genuine opportunity shall serve as an occasion to identify the challenges in applying current engineering concepts to various types of historic buildings, and also to learn the classical construction methods and share ideas for future projects.

Our success during this workshop will depend on the ideas and enthusiasm brought by the participants. In that sense, the participants are encouraged to actively participate in discussions and in the process of developing a resolution.

To the end, I would like to express my sincere appreciation to the DPRI for the generous financial support and the administrative staffs for making this opportunity possible.

Sincerely yours,

Masahiro Kurata, Ph.D.

Assistant Professor and Program Coordinator
Division of Earthquake Research
Disaster Prevention Research Institute (DPRI), Kyoto University
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International Workshop by Young Researchers for "Application of Structural Engineering and Structural Health Monitoring to Historic Buildings"

Friday, December 19th (Room S-519)			
8:50–9:00	***Registration***	11:45–13:00	***Lunch (Cafeteria)***
9:00–9:10	***Opening***: Masahiro Kurata, DPRI @ Kyoto University	***Afternoon Session 1: Structural Health Monitoring Techniques***	
Morning Session 1: Structural Engineering Application to Historic Buildings		13:00–13:20	Akiko Suzuki, Kyoto Univ. Damage Assessment of Steel Beam-Column Connections using Ambient-Based Inner-Force Estimates
9:10–9:35	Miho Sato and Shohei Shimmoto, Kyoto Univ. Restoration of Chu-Kondo Main Hall in Kofukuji Temple	13:20–13:40	Li Xiaofua, Kyoto Univ. Damage Detection of Beam Fractures in Steel Buildings under Earthquake Loading
9:35–10:00	Diana Faiella, Univ. of Naples, Italy Seismic Behavior of Masonry Churches in 2009 L'Aquila EQ: Damage Survey and Numerical Assessment	***Afternoon Session 2: Advancements in Structural Design and Analysis***	
10:00–10:25	Hiroyuki Inamasu and Hiromichi Nishino, Kyoto Univ. Retrofit of Toshodaiji Temple, Golden Hall	13:40–14:05	Francesca Barbagallo, Univ. of Catania, Italy Preliminary Validation of a Multimodal Adaptive Procedure
Morning Session 2: Seismic Protective Systems		14:05–14:30	Konstantinos Skalomenos, Univ. of Patras Estimation of Seismic Drift and Ductility Demands in Composite Framed Structures: A Design Approach
10:30–10:55	David Figueiredo, Univ. of São Paulo, Brazil Lumped Damage Mechanics as an Alternative to Analyse Masonry arches	14:30–14:50	***Coffee Break***
10:55–11:20	Zhang Lei and Ikumi Hamashima, Kyoto University Seismic Rehabilitation Techniques	14:50–16:30	***Discussion and Resolution Making***
11:20–11:45	Liusheng He and Takuma Togo, Kyoto Univ. Steel Shear Walls with Double-Tapered Links Capable of Condition Assessment	16:30–16:40	***Closing Remarks***: Masahiro Kurata, DPRI @ Kyoto Univ.

Notes: Each presentation shall include 5 min for question and answer.

BIOGRAPHIES AND ABSTRACTS

Miho Sato
Graduate Student
Master of Architectural System, Kyoto University

Miho Sato is a Master student at the department of architecture and architectural engineering in Kyoto University. She obtained her Bachelor of Architectural Engineering at Kyoto University in this March under the guidance of Professor Nakashima and Professor Kurata. For her Bachelor’s study on the development of rehabilitation techniques for steel structures, she received an award for the outstanding graduation thesis by Architectural Institute of Japan. In her Master’s study, she is currently working on the application of the developed system to low-rise steel frames and examining the effectiveness in enhancing seismic performance.

Shota Shimmoto
Undergraduate Student
Master of Architectural System, Kyoto University

Shota Shimmoto is an undergraduate student of the school of architecture at Kyoto University. Mr. Shimmoto graduated Shimonoseki Nishi High school and entered Kyoto University in 2011. He just started his research in last September and is working for his graduation thesis. His research subject is decision making after earthquake and his advisor is Prof. Kurata. His work includes shaking table collapse test of the three story building model and development of a decision-support system for damaged building.

Title:
Concept and Proceedings of Restoration Program for Chukon-do hall in Kofukuji Temple

Abstract:
Kofukuji-temple in Nara prefecture was originally established in A.D. 714. Kofukuji was tutelary temple of Fujiwara clan, which was a powerful family of regents in Japan. The main hall of Kofukuji was made of timbers and lost by fire for seven times. Until the 14th century when Fujiwara clan maintained their power, the main hall was rebuilt to its original shape not so long after they were damaged. However, after the fire in 1717, the original hall had never restored to date.

The restoration project of the large hall was launched in 1998. Since the hall had been lost for long time, the project started from surveying its original appearance and structural features by excavation and literature. The construction of the hall finally started in 2008 and is supposed to be completed in 2018. The construction process can be divided into four parts: (1) construction for foundations; (2) temporal covering; (3) erection of timber structure; and (4) seismic retrofitting. In general all the restoration work finish once the timber structure is built, however, due to the strict Japanese structural standard, the additional seismic retrofitting was required to satisfy the criteria. An inelastic incremental analysis and time history analysis were conducted to verity the seismic capacity of each element and the entire building.

Restoration of Chu-Kondo Main Hall in Kofukuji Temple

Hajime Tateishi[1], Miho Sato[2], and Shota Shinmoto[2]
[1]Tateishi Structural Engineering Company, Kyoto, Japan
[2]Department of Architecture and Architectural Engineering, Kyoto University, Kyoto, Japan

Kofukuji-temple in Nara prefecture was originally established in A.D. 714. Kofukuji was tutelary temple of Fujiwara clan, which was a powerful family of regents in Japan. The main hall of Kofukuji was made of timbers and lost by fire for seven times. Until the 14th century when Fujiwara clan maintained their power, the main hall was rebuilt to its original shape not so long after they were damaged. However, after the fire in 1717, the original hall had never restored to date.

Fig. 1 Estimated original appearance of Chukon-do.

The restoration project of the large hall was launched in 1998. Since the hall had been lost for long time, the project started from surveying its original appearance and structural features by excavation and literature. Fig 1 shows the estimated appearance. The construction of the hall started in 2008 is supposed to finish in 2018.

The foundation stone was built at the original location (Fig. 2). To protect the original foundation which is considered as cultural heritage, the foundation was constructed in two steps: the protective layer made of sand was placed to cover the original foundation and; the high-strength concrete slab with the holes to emit water was placed. The protective sand layer was incorporated to save the platform, because the platform was designated as a historical landmark. Next, the temporal cover was built to protect the building under construction from rain or wind, and to improve the effectiveness of construction. The huge space was generated using the latticed structure with lots of steel pipes. Once the temporal covering finished, the construction of entire wooden structure began. Following the manner of Japanese hysterical buildings, all the elements were combined without any steel bolts or nails, but with traditional configuration of wooden parts. Even structurally important lateral component was also just inserted to column without pasting or fixing (Fig. 3). Size of the hall ended up with 36.6 m times 23 m with the maximum height of 21.2 m, as the estimated original structure.

In general all the restoration works may be completed once the timber structure is built, however, the Japanese building law required the additional seismic retrofitting to satisfy seismic design criteria. The criteria suggested by Japanese architectural center were as follows: (1) damage limit: the story drift should be within 1/60 and all the elements should basically remain elastic, with the input which had the maximum velocity of 10 m/s; (2) safety limit: the story drift should be within 1/30 and all the elements should not reach to its ultimate strength. An inelastic incremental analysis and time history analysis using 3D frame (Fig. 4) were conducted to verity the seismic capacity of each element and whole building could exceed the boundary enacted by the standard. The 3D analysis model strictly reflects the real condition as frame geometry, mass, material, size and restoring force characteristics for all the structural elements. According to the result of these analyses, the hall could satisfied the design requirements, with some additional retrofitting techniques: adding shear wall consisted of wooden grid and steel frame, connecting lateral component at the top of column by metal straps and bolts; attaching structural plywood to the ceiling of nave and pent roof; installing steel braces to wooden roof truss.

Fig. 2: Foundations.

Fig. 3: Timber frame erection.

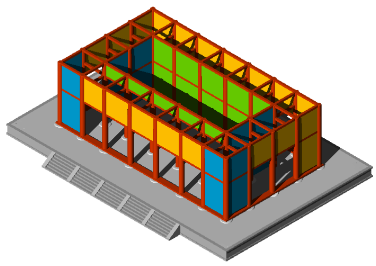

Fig. 4: 3D modeling

Diana Faiella
Ph.D Student
Department of Structures for Engineering and Architecture,
University of Naples "Federico II", Naples, Italy

Diana Faiella is a Ph.D student in "Structural Engineering and Seismic Risk" at the University of Naples Federico II. She completed her graduate studies in September 2014 and started the Ph.D program in November 2014. The topic of her Master thesis concerned the seismic isolation, and basically was a knowledge acquisition study of the advanced Japanese design practice; the thesis started from the collection of data concerning the structural properties of seismic isolated buildings and the relative design criteria, as well as of data registered in actual seismic events on isolated buildings. The data have been processed and statistically analyzed in order to: (1) derive trends in current design practice and relevant criteria; (2) assess the efficiency of isolation system. Further the thesis has focused on a specific case study – a long span structure, "Main Building of the Shimizu Corporation", Tokyo –for which several alternative solutions for the isolation system have been developed and evaluated in term of costs and performance parameters.

In the course of the Ph.D Program she will continue to work on seismic isolation issues, also contributing to a research project focused to failure modes and relevant design criteria for HDRBs. In addiction she has been currently involved in the research activity on seismic behaviour and assessment of ancient masonry structures , which is being carried out starting from the late 1990s' by the research group of Prof. Mele and De Luca

Title:
SEISMIC BEHAVIOUR OF MASONRY CHURCHES IN 2009 L'AQUILA EARTHQUAKE: damage survey and numerical assessment

SEISMIC BEHAVIOUR OF MASONRY CHURCHES IN 2009 L'AQUILA EARTHQUAKE: damage survey and numerical assessment

Giuseppe Brandonisio, Giuseppe Lucibello, Antonello De Luca, Diana Faiella, Elena Mele
Department of Structures for Engineering and Architecture, University of Naples "Federico II", Naples, Italy

ABSTRACT

The seismic behaviour of masonry churches damaged during the 2009 L'Aquila earthquake is studied in this paper. Four important basilicas are considered in order to derive general conclusions from the damage assessment and the performance analysis. As a general result of the comparison between the post-earthquake survey activity and the structural analyses the possibility of evaluating the seismic safety of churches, and therefore of avoiding destructive damage by means of the design and application of appropriate retrofit interventions, is confirmed.

Comparative numerical analyses on a sample of four churches have highlighted another important aspect: the dynamic excitation due to the seismic ground motion activates many vibration modes of the building structure, though all of them are characterized by small participation factors. This fact while confirms the validity of the macro-elements approach, also leads to the following important consequences: the high spectral values of the registered record of the L'Aquila earthquake do not correspond to equivalent high values of base shear; in particular the results showed that in all the examined case studies, the base shear V ratio ranged between 20% and 30% of the church weight. Therefore the appropriate choice of the force reduction factor to be adopted for these monumental buildings is not so important as for new buildings, since the real shear force value was significantly smaller than the plateau value of the spectral acceleration provided by Italian Code. Furthermore, the awareness of the activation of many local modes under seismic excitation calls for retrofit interventions which have to ''tie up'' the building, thus avoiding local failures that are often observed.

The final conclusion is that the observation of damage and failures under real experimental actions, like real earthquakes, are a precious means for the advancement of knowledge in the field of seismic engineering.

KEYWORDS

Masonry buildings, Seismic damage, Non-linear Analysis, Limit Analysis

INTRODUCTION

The current paper is a very brief summary of the paper [1].

Basilica structures are usually constituted by a façade, a hall (with one or more naves), a presbytery and an apse; besides, a transept, a dome and lateral chapels can be added; often, a bell tower is present. More in general, churches are characterized by the presence of long and slender wall panels without internal bracing walls, presence of thrusting elements of large span (arches, vaults and domes), lack of intermediate connections, degradation due to the ancientness and poor maintenance. The high seismic vulnerability of church heritage, as demonstrated at earthquake occurrence, is mainly related to the specific building configuration, as well as to the mechanical properties of the masonry material, characterized by highly non-linear behaviour and very low tensile strength. The assessment of the seismic vulnerability is of primary importance in order to preserve the integrity, the usage, historical evidence and architectural value of this important portion of our cultural heritage.

MODELLING AND ANALYSES

The rigorous assessment of the seismic behaviour of complex masonry buildings shows objective difficulties due to several reasons: firstly, the analysis of the masonry materials, characterized by non-linear behaviour and low tensile strength, requires complex theoretical modelling, usually not straightforward to be implemented in a finite element model. Secondly, the arrangement of blocks and mortar joints in the structural elements is frequently uncertain and variable; as a result, the mechanical properties of the masonry material may show significant scatters throughout the building, and the experimental characterization very often implies simplifications, which can mislead the real behaviour.

Finally, additional difficulties are related to the highly composite geometry and morphology, which drive to three-dimensional (3D) models characterized by a large number of degrees of freedom.
Within this framework a two-steps procedure for the seismic analysis of basilica churches was proposed by the authors [2] and applied to different basilica churches [3]. In the first step of the procedure, the church building is analyzed in the linear range with 3D finite element models, in order to determine the static and dynamic properties, and the seismic demand, i.e.: the distribution of horizontal force acting on each constituting parts, the so-called macro-elements, which are characterized by autonomous structural behaviour under seismic loads. In the second step the complex 3D structure is dissected in the constituting macro-elements, which are separately analyzed up to collapse through either refined non-linear 2D models and/or limit analysis, in order to evaluate the seismic capacity, i.e. the horizontal strength of each macro-element.
The results from the two steps, expressed in terms of seismic demand and seismic capacity, are then compared with the purpose of assessing, though in an approximate way, the safety level of each macro-element of the global structure and provides indications on the susceptibility of the church to seismic damage and partial or total collapse.

CASE STUDIES

Four Aquilan basilica churches, Santa Giusta (SG), Santa Maria di Collemaggio (SMC), San Pietro di Coppito (SPC) and San Silvestro (SS) are analysed in this paper . In Fig.1 are provided the plans of the churches, with identification of the macro-elements in both transversal and longitudinal directions. SG is a typical church characterized by a central nave and lateral chapels, while SMC and SS are typical basilica churches with three naves; on the contrary the plan of SPC church is non-typical, because it is characterized by a short and almost square hall, and a transept of comparable size.

All the churches were built in the same period, i.e. XIV century. In general their bearing masonry structure is realized by using the sack masonry arrangement, typical of L'Aquila region, and/or freestone. The roof structures are made by either wood trusses and/or masonry vaults. SPC has one of the rare bell towers in the city of L'Aquila.

A detailed survey in the aftermath of 2009 earthquake revealed a widespread damage in the churches, schematically reported in the Figs.2-5.

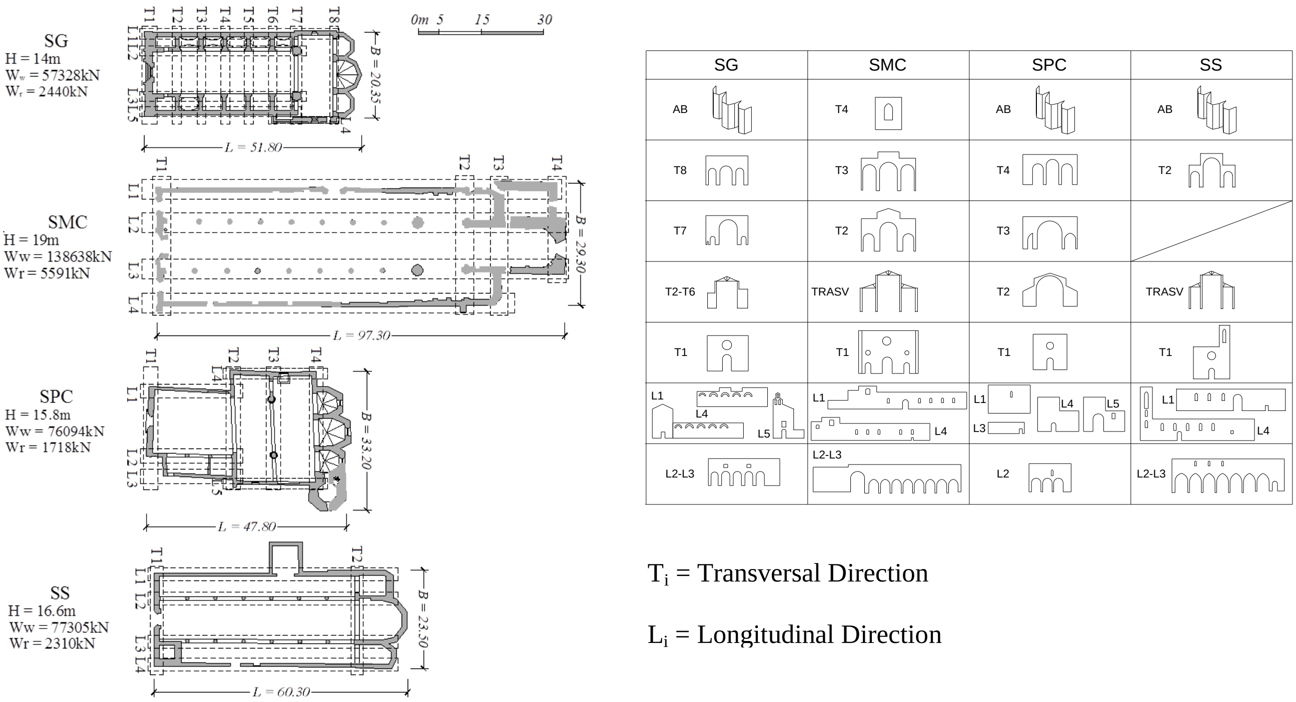

Figure 1 Linearized plans of churches, global dimensions (B, L, H), weight (W_w=weight of walls, W_r=weight of roof), identification of transversal (Ti) and longitudinal (Li) macro-elements.

Performance evaluation through the two step procedure

First step: the demand on the macro-elements
Modal dynamic analyses were carried out by using both the response spectrum obtained from the acceleration history recorded at the AQK station, that is located in L'Aquila city, and the design spectra (elastic and inelastic) suggested by the Italian Building Code [4], compared in Fig.6a.

Since the dynamic behaviour of the four churches is quite similar, some major results are briefly reported in the following only for SG. The distribution of vibration modes of SG, provided in Fig.6b, shows that almost all vibration modes (for the first 100 ones) have modal participating mass ratio (M_{eff}) less than 10% and even lower ratios at peak acceleration periods. The base shear V (according to both the AQK and NTC'08 spectra), evaluated using the SRSS combination of maximum modal responses and normalized to the total building weight (W_{tot}) in both the transversal and longitudinal directions, ranges between 20% and 30% in all the four case studies.

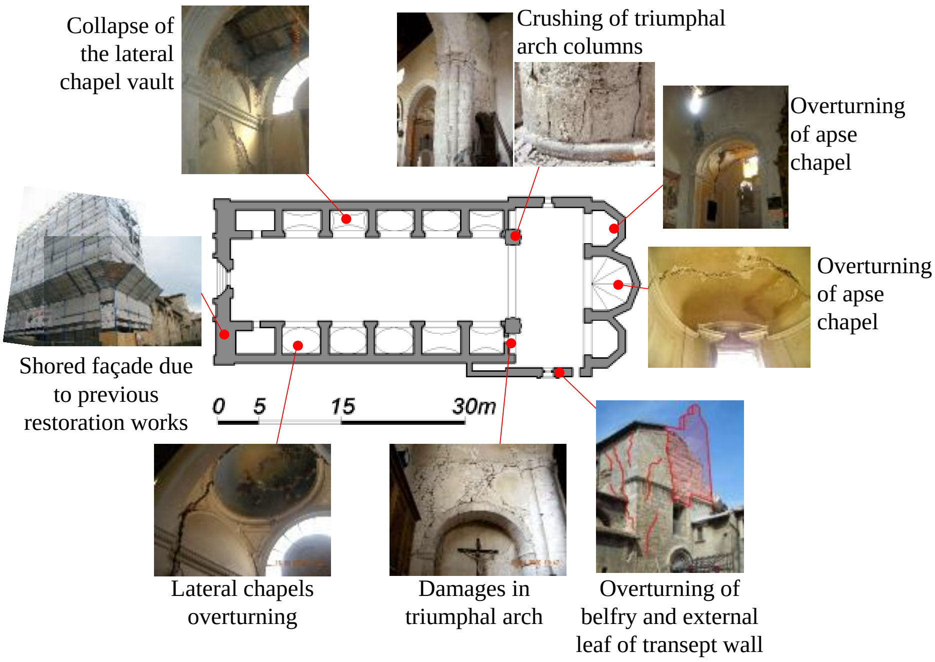

Figure 2 Santa Giusta church (SG): damage after the 6th April 2009 earthquake.

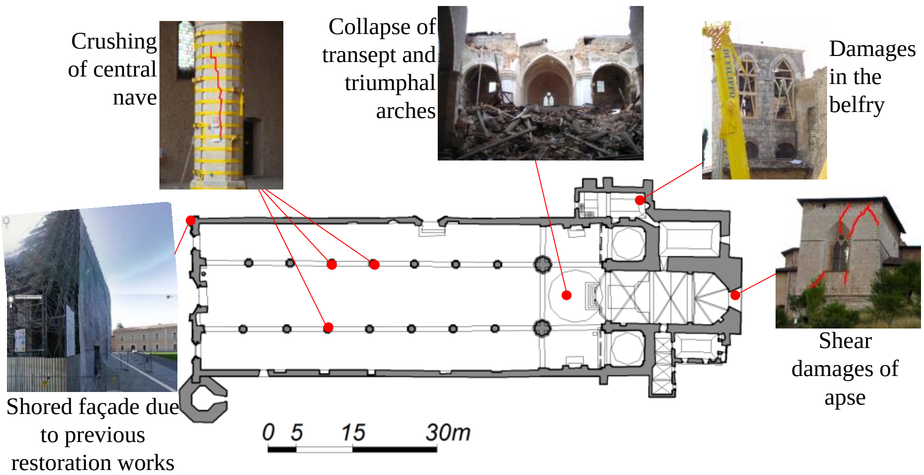

Figure 3 Santa Maria di Collemaggio church (SMC): damage after the 6th April 2009 earthquake.

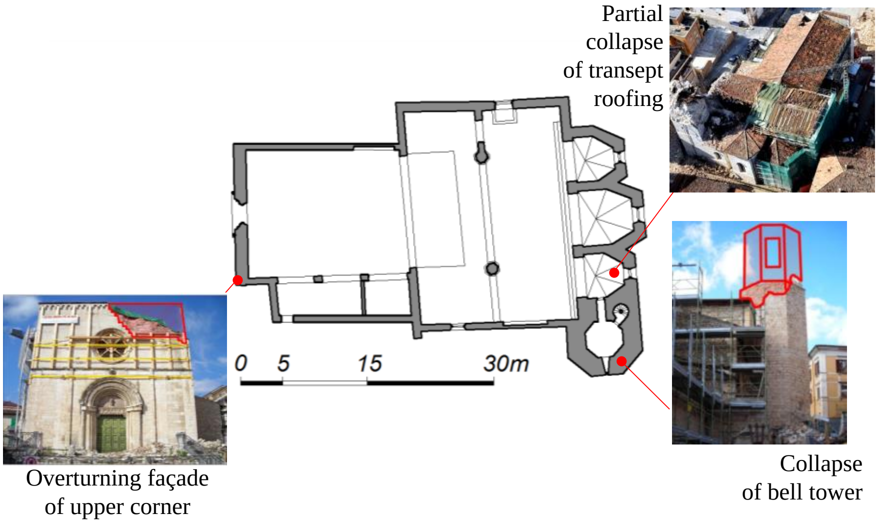

Figure 4 San Pietro di Coppito church (SPC): damage after the 6th April 2009 earthquake.

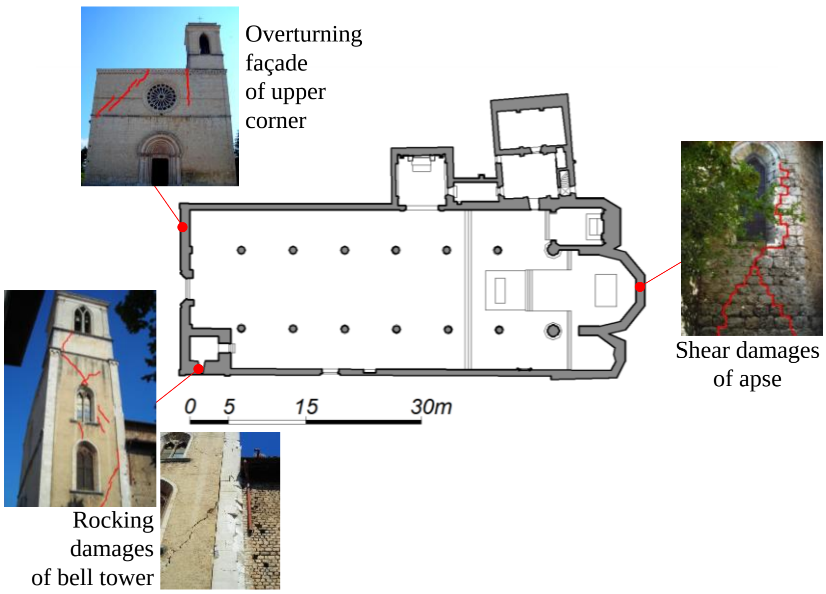

Figure 5 San Silvestro church (SS): damage after the 6th April 2009 earthquake.

The distribution of the base shear among the single macro-elements (V_i/V) has been computed for both transversal and longitudinal seismic action. In the first case the shear force is mainly concentrated in the perimeter and transept elements (10-30% and 5-10%) while in the second case in the internal ones (5%).
Concerning the percentage of seismic base shear globally absorbed by the macro-elements located in the direction orthogonal to the applied seismic action ($V_{out\text{-}of\text{-}plane}/V$), it can be observed that a larger contribution of orthogonal elements arises when the seismic action is applied in the transversal direction (25-75%) than in the longitudinal one (10-30%).

Second step: the capacity of the macro-elements
The non-linear analyses were carried out by applying constant gravity loads and by increasing the horizontal loads, proportional to the masses, up to the structural collapse. For this aim, the finite element computer code ABAQUS was used; the material non-linearity and in particular the nearly no-tension characteristics of the masonry were accounted for by means of a smeared crack model. A compressive strength equal to 1.0 MPa and a tensile strength equal to 0.1 MPa were adopted, with tensile-to-compressive strength ratio assuming the classic value 1/10.

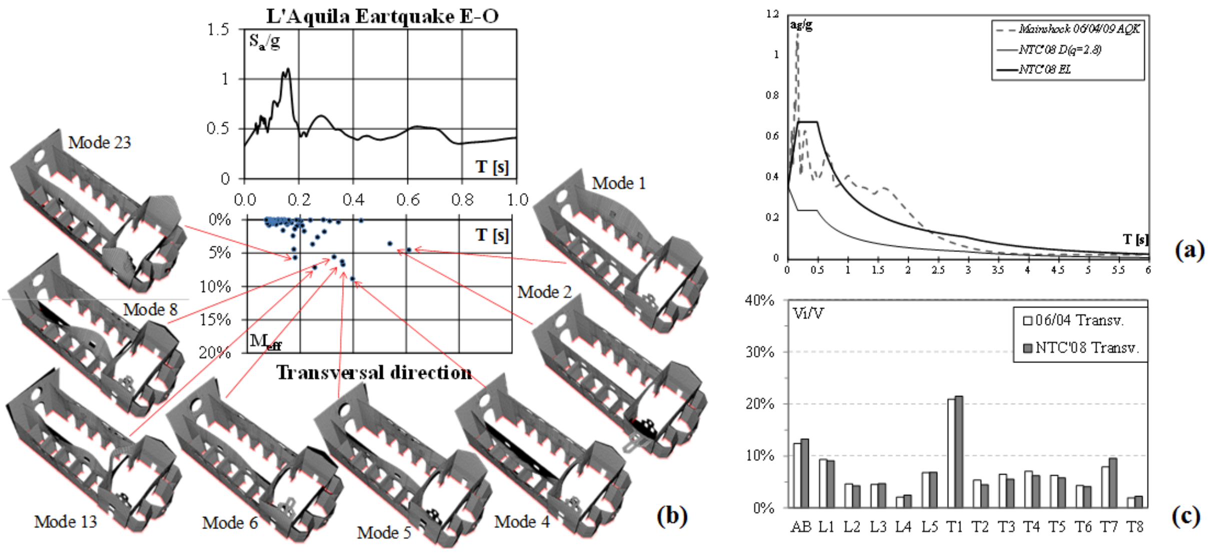

Figure 6 (a) AQK Response Spectrum and Code Design Spectra, (b) SG church: Modal Shape and Participating Mass Ratios, (c) SG church: Shear Distribution among macro-elements in transversal direction

The results of the non-linear analyses on the single structural macro-elements allow for obtaining push-over curves, stress and plastic strain distributions, deformation and collapse modes, and the ultimate horizontal strength capacity. In the following, for the sake of brevity, only the values of ultimate lateral strength (V_u) are discussed, particularly in comparison with the strength demand (V_i) obtained for each macro-element in the first step of the procedure.

In Fig.7 this approximate assessment of the churches under the main shock 06/04/09 AQK is presented for all the macro-elements of the church case studies, by depicting the demand-to-capacity ratios (V_i/V_u) as bullet points in the relevant chart.

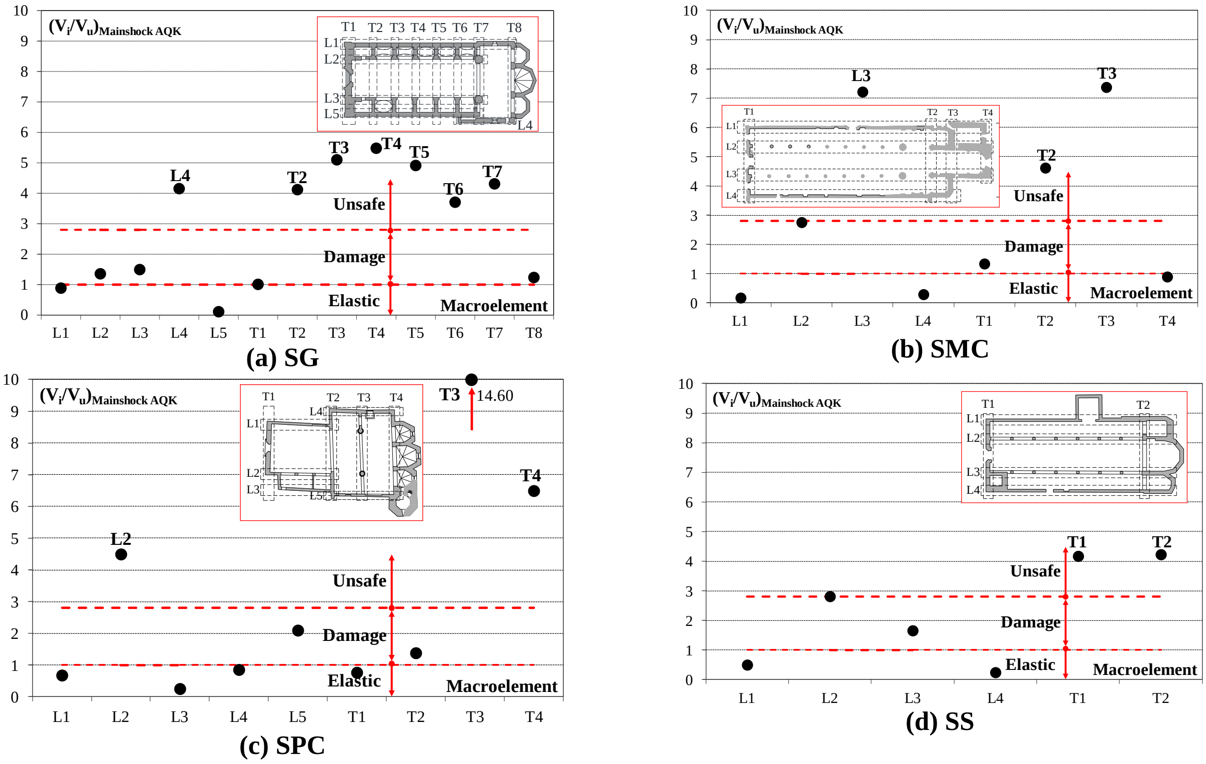

Figure 7 Horizontal seismic demand (V_i) to strength capacity (V_u) ratio for the macro-elements.

The horizontal lines $V_i/V_u = 1$, corresponding to the elastic limit condition and the one at $V_i/V_u = 2.8$, which represents a level of damage calculated by using a behaviour factor of q=2.8, are also plotted. It can be noted that, with the exception of few perimeter elements, the macro-elements of the churches usually have a strength capacity smaller than the demand counterpart ($V_i/V_u > 1$).

For assessment of the out-of-plane response of masonry structures, limit analysis is among the most straightforward, effective and stable procedures. The application of limit analysis, through a kinematic approach, requires the identification and the analysis of all collapse mechanisms that can be activated in the building. Considering the façade macro-element, four collapse mechanisms have been identified: global overturning, partial overturning of the façade upper part (gable); horizontal bending of the gable; overturning of the façade upper corner. For each collapse mechanism, the seismic acceleration acting on the portions of rigid bodies involved in the kinematism Δa is compared to the acceleration value that causes activation of the collapse mechanism a_0^*. The results obtained from the out-of-plane kinematic analysis reported in Fig.8 suggest that the façade of SS, SG and SPC churches are highly susceptible to the overturning mechanisms of the upper corner, as also observed after the earthquake (Fig.8), with values of $\Delta a/a_0^*$ equal to 1.17, 1.23 and 1.76 respectively.

Comparison between two-step procedure results and seismic damage

A good agreement between the analysis results and the observed damage has been found, not only in terms of ultimate base shear but also in terms of collapse mechanisms and crack patterns, as already shown in Figs. 8 and in the following Figs9-10. As an example, in the SG church it is evident that the macro-element T7 (triumphal arch) and L4 (transept wall) are highly vulnerable with the demand-to-capacity ratio (V_i/V_u) greater than 4; in fact, after the 2009 earthquake crushing failures in these macro-elements were observed. In the case of SMC the most vulnerable macro-elements are the central arcades (L2 and L3) and the first and second triumphal arches (T2 and T3) with (V_i/V_u) equal to 3, 7, 5 and 7 respectively.

They were actually the most severely damaged parts and some columns collapsed due to combined bending and axial load triggering the whole collapse of the transept.

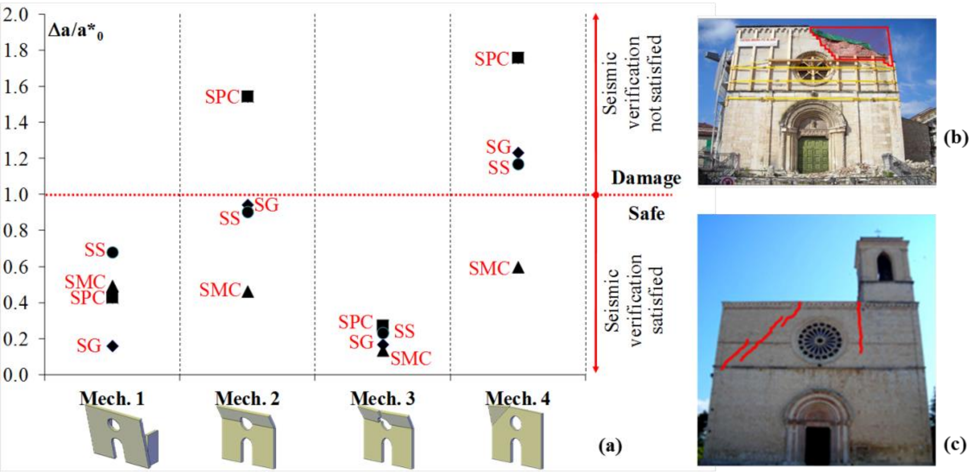

Figure 8 (a) Out-of-plane collapse of façades (b) San Pietro a Coppito church (SPC) and (c) San Silvestro church (SS)

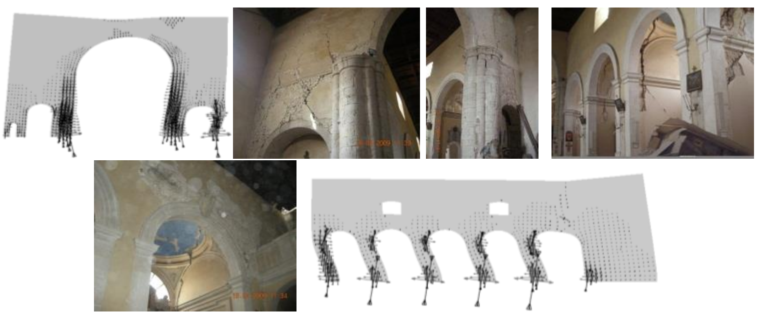

Figure 9 SG - seismic damage and analysis results: triumphal arch (T7) and arcade (L3).

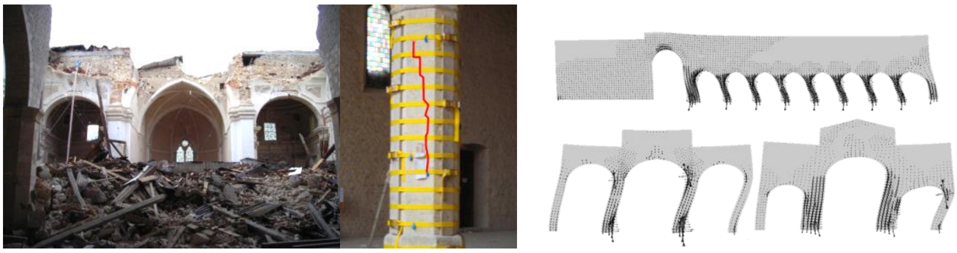

Figure 10 SMC - Seismic damage and analysis results: arcades (L2 and L3) and triumphal arches (T2 and T3).

CONCLUSIONS

The results of the comparison between the post-earthquake survey activity and the numerical analysis on the church case studies confirms that, nowadays, it is possible to evaluate the seismic safety level of churches and to identify the potential failure modes; this, in turn, allows monumental heritage to be preserved and destructive damage to be avoided by adopting appropriate retrofit interventions. In particular, the reliability of the two-steps analysis, suggested and applied by the authors for predicting real behaviour of churches under seismic actions, is confirmed. Other important results concerning the seismic behaviour of masonry churches have been obtained: the dynamic excitation due to the seismic ground motion activates many vibration modes of the building structure, though all of them are characterized by small participation factors, generally less than 10%; for this reason the high spectral values of the registered record of the L'Aquila earthquake do not correspond to equivalent high values of base shear on the churches. In particular the results showed that in all the examined case studies, the base shear V ranged between 20% and 30% of the church weight W_{tot}. Therefore, as the global shear force on the buildings was significantly smaller than the plateau value of the spectral acceleration provided by Italian Code, the appropriate choice of the force reduction factor to be adopted for these monumental buildings is not as significant as in the case of traditional residential buildings characterized by shear type behaviour. Further, the activation of many local modes also calls for retrofit interventions which should ''tie up'' the building, thus avoiding the local failure modes that are often observed. The final conclusion is that use of data coming from the ''Natural Laboratory of Earth'' is the best approach to evaluate the seismic behaviour and to calibrate assessment procedures, and is therefore an essential step for the advancement of knowledge in the field of seismic engineering.

ACKNOWLEDGMENTS

The authors gratefully acknowledge the precious contribution of Dr. Aldo Giordano, eng. Giuseppe Fappiano, Dr. Rosa de Lucia and Dr. Roberta Santaniello in the post-earthquake reconnaissance activity.
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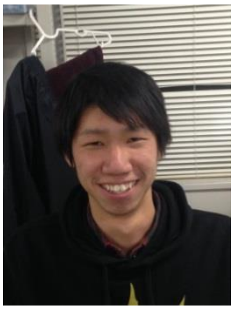

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Title:
Retrofit and Seismic Assessment of Historical Wood Building "The Main Hall of Toshodaiji-Temple"

Retrofit and Seismic Assessment of Historical Wood Building "The Main Hall of ToshodaijiTemple"

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ABSTRACT

A national historic site "Toshodaiji-temple" was founded by a Chinese Buddhist monk named Jianzhen about 13 centuries ago. The main hall registered as a national treasure is considered the archetype of classic Japanese building style while it experienced several major retrofit. This report presents the latest and modern retrofit project which also included seismic assessment. The project focused on to correct columns leaning toward inner side due to long-term gravity and to examine the seismic performance and load-resisting mechanism of Japanese traditional wooden architecture through a modern numerical study. The property of wood materials depends on the load direction against fibers. Fiber direction has larger strength and stiffness than the normal direction to fiber. On contrary the normal direction is more ductile than fiber direction. Therefore, the consideration on the fiber direction is the key in classical wood construction. Other remarkable components of the traditional wooden architecture are follows: foundation stone; halving joint; penetrating tie beam, self-righting column. At the base, columns are not fixed to the ground and thus the building behaves like a rocking system under earthquake loading. At the joint, members are just joined each other without connecting, which is called a halving joint. In the numerical evaluation, to simulate column behavior, penetration at the bottom surface and rotation of joint support at the column top was considered in the analysis model. Using an analysis model which considered all the above features, earthquake response analysis and gravity load analysis were conducted. In the earthquake response analysis, three major ground motions and two local-site ground motions were used. Results showed that the maximum response was about 1/25 story drift. In the gravity load analysis, the mechanism of column leaning was clarified. Based on the analysis result, the retrofit strategy was determined. In the retrofit, roof mass was reduced and knee brace was additionally installed. Thanks to the two major reinforcements, column-leaning became 20 times smaller than before the retrofit.

KEYWORDS

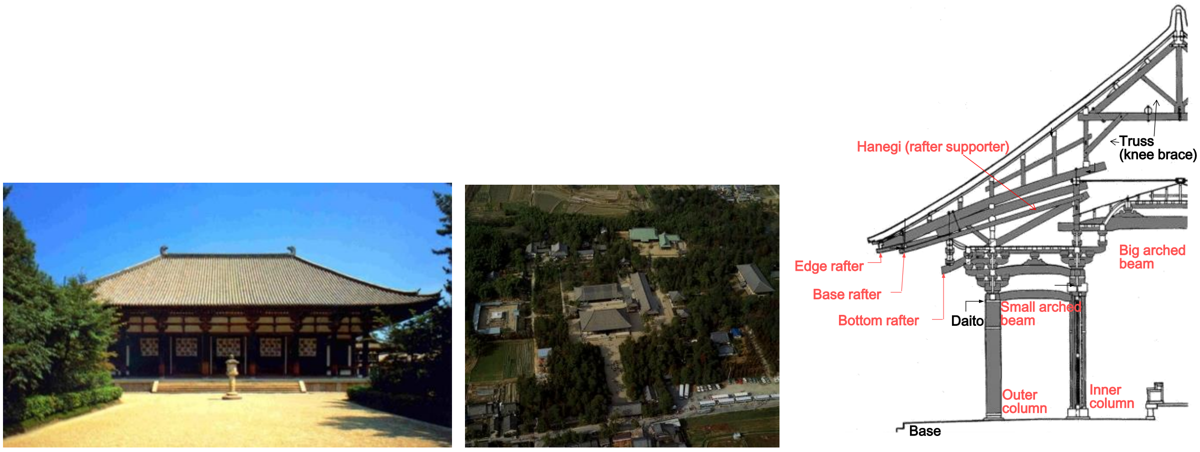
Japanese traditional wood structure; seismic assessment; retrofit; analysis model

INTRODUCTION

Japan has many historical wood architectures. As a national historic site, "Toshodai-ji temple" in Nara prefecture has many building registered as a national treasure. The temple was founded by a Chinese Buddhist monk named Jianzhen about 13 centuries ago. To keep its condition, many major retrofitting had been conducted to date. At each time, different type of retrofitting was adopted because this kind of structure has several unique construction methods as shown fig.1(c). The retrofitting of historic structures are always challenging since changing to original plan, elevation and configuration detail are not preferred and supplemental member shall be removable and invisible from visitors. Restriction is even severe for national treasures.

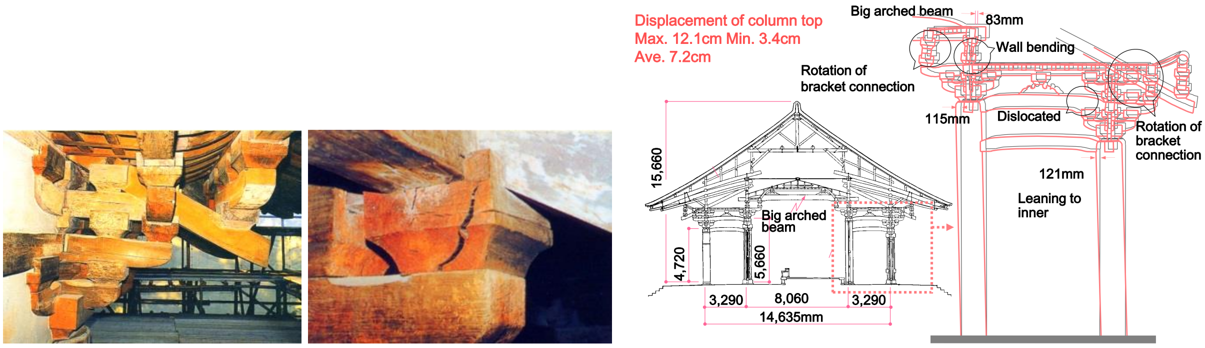
In 2009, Takenaka co. was asked to retrofit the main hall of the "Toshodaji-temple", called "Kondo", and examine its seismic performance (fig. 1(a)). The main hall locates at the center of the view in fig. 1(b). As shown in fig.2(a) and (b), the joints of the Main hall was rotating and crack appeared in some components. Moreover, the inner columns were leaning toward inner side with rotating joints as shown in fig.2 (c). The maximum lateral displacement of column top was 12.1cm, minimum was 3.4cm and average was 7.2cm. This is due to deformation and rotation due to several members. For example, rotation of bracket connection, dislocating of big arched beam shape and rotation of column itself. These deformations might be because of gravity load and wood aging. To solve this problem, several retrofitting methods were planned. To select the most appropriate method of retrofitting, Takenaka corporation did a lot of testing and developed an analysis model for typical Japanese traditional wood architecture.

In this project, there were two objectives. First, column leaning was corrected by deploying an additional resisting system to gravity load. There, reasonable and the most appropriate method of retrofitting was considered. Second, an analysis model which captured the load-resisting mechanism of Japanese traditional architecture was constructed. With the developed model, the seismic performance and long-term resistance were examined through earthquake response analysis and gravity load analysis. This report presents key points in the analysis model and the results of earthquake response analysis and gravity load analysis.



(a) Main hall (b) General view (c) Main hall section

Figure 1 Toshodaiji-temple



(a) Rotation of joint (b) Crack of component (c) Inner column leaning

Figure 2 Damage in the Main hall

ANALYSIS MODEL

This section introduces the material property and remarkable structural components of Main hall which were considered in the analysis model.

i. Material

Wood material has different material property in load directions (Fig.3). When load is applied parallel to the fiber direction it shows large strength and stiffness but relatively brittle. On contrary, in normal to fiber direction, wood is ductile with relatively small strength and stiffness. The strength in the parallel direction is 3~8 times larger than that in the normal direction, and the stiffness in the parallel direction is 10~25 times larger than that in vertical direction. Moreover, penetration to material often occurs in the normal direction and at member level, this phenomena provides additional stiffness. Therefore, the direction of wood member must be considered well in each part.

In this project, after disassembling all members, several tests such as bending and axial loading tests were conducted to examine the material property in Main hall. Measurement of Young’s modulus was conducted before disassembling using a technology developed and named ‘Hammering experiment’ by Takenaka corporation. The material property from these tests was implemented into the analysis model. Moreover, it was found that the strength of members used was larger than that of new members to be replaced. This was a typical feature for a kind of wood, ‘Japanese cypress (Hinoki)’.

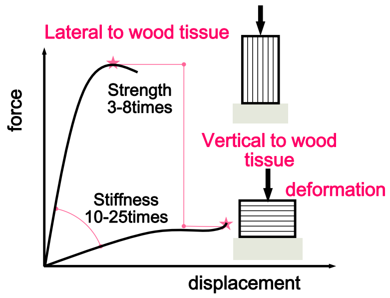


Figure 3 Material property of wood material

ii. Components

1. Foundation stone (Base)

Generally, current wood buildings are mandated to be fixed to the base for resisting to seismic force, but traditional wood architecture has special boundary condition. Fig.4 shows the details of so-called foundation stone. It has hollow-out surface, and the bottom part is buried in the ground. Column is just placed on this stone, but stable because of the hollow-out surface. Thus, the column behaves like ‘rocking system’. This feature is considered in the boundary condition of the analysis model.

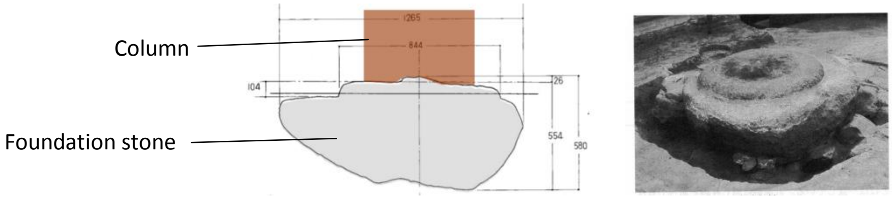


Figure 4 Detail of foundation stone

2. Halving joint

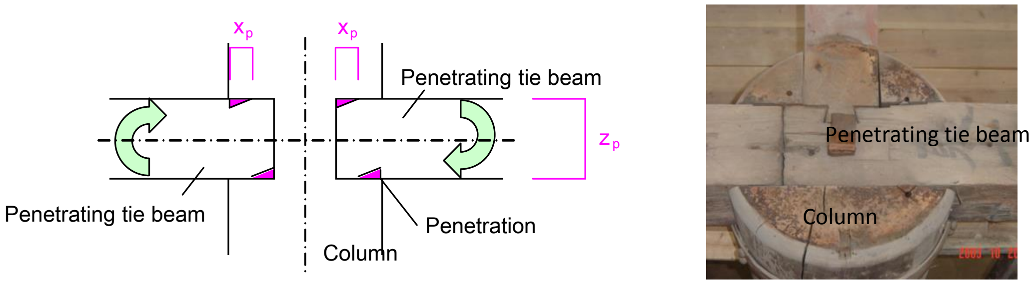
Japanese traditional architecture has relatively flexible connections because at no connecters like nails or bolts are used. To joint members each other, the sections of each member section is properly cut, and then, each cut section is assembled. This kind of joint is called ‘Halving joint’. Fig.5 (c) shows the mechanism of stiffness reduction against gravity load in so-called bracket-arm joint. Compared to the bracket before cut, it is easier for the cut bracket arm to deform in vertical direction. Therefore, equivalent stiffness (equivalent α) considering cut should be calculated and used in the analysis model. In addition to stiffness reduction, to simulate connection rotating, penetration for normal direction to wood fiber should be considered. In the entire building model, there are various types of joints. The stiffness and rotation of all type of connections are estimated and considered to the model in all building directions.

(a) Member in connection (b) Joints (c) Mechanism of stiffness reduction

Figure 5 Detail of halving joint

3. Penetrating tie beam

Fig.6 shows the details of penetrating tie beam. Penetrating tie beam behaves similar to halving joint. Here, penetration is also considered at the beam edge. As shown in fig.6 (a), penetration occurs at the edge when the bending moment is applied to beam. To calculate the rotational stiffness for this connection, calculation methods are developed.



(a) Penetrating location (b) Section of connection

Figure 6 Detail of penetrating tie beam

4. Self-righting column

Column doesn't resist to bending moment because it's not fixed to the base as explained in the section for foundation stone section. Instead, traditional architecture resists in rocking. Rocking resistance is also called self-righting system in which resisting force is obtained from gravity load. Fig.7 shows the mechanism of self-righting column. Until left side of column top rotating beyond the right side of column bottom, structure returns to its original location. In most cases, traditional structures adopt wide columns, which have large rotation capacities, and roof mass is relatively large with stone tile. These two features enable the column to have large rocking resistance capacity. When considering the retrofitting, you can see that reducing roof mass is not necessarily good for the structure although inertia force is reduced, because it means lateral stiffness reduction of column.

If the column is assumed to rigid, the column would behave like the mechanism shown in fig.7. However, actually, the effects of penetration at the column bottom and the connection rotation at the column top also contribute to lateral stiffness of column. In the analysis model, column with these features are preferred.

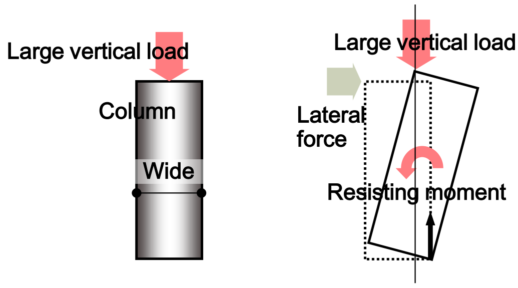


Figure 7 Mechanism of self-righting column

ANALYSIS RESULTS

Analysis model was developed by considering aforementioned features For gravity analysis, a half of entire building was analyzed. In total, the model had 8000 nodes and 17000 elements. For earthquake response analysis, entire building model was analyzed. The model had 15000 nodes and 21000 elements.

i. For ground accelerations

First, the vibration characteristics were examined. The vibration modes of 'Main hall' was estimated as follows; the 1st mode with T1=0.91s for the long edge direction, the 2nd mode with T2=0.83s in torsion, the 3rd mode with T3=0.63s for the short edge direction. In the earthquake response analysis, five ground motions were used. Three ground motions were major ground motions, El Centro, Taft and Hachinohe. Two ground motions were synthesized local ground motions for future Ikoma fault and Nankai trough earthquakes. The maximum roof drifts are summarized in table 1. Fig. 8 shows the deformed shape of the structure under Taft ground motion. The maximum response among all input ground motions was about 4% in story drift (before retrofitting), which meant Japanese traditional architecture had large deformation capacity.

ii. For gravity load

From the gravity load analysis result, the mechanism of column leaning to inner was examined. Fig. 9 shows the force transmission in the structure. Gravity force transmitted to the outer column through roof. Then, gravity force was replaced to lateral force because roof pushed the column from outer side. However, in the frame before retrofitting, there was no resisting force to this lateral force because of the rotation of connection at the inner column top and member aging. Therefore, in retrofitting, it was aimed to transmit the central gravity force to the inner columns.

Table.1 Results of earthquake response analysis

Input ground motion	For longer edge dir.	For shorter edge dir.
El Centro	1/34	1/29
Taft	1/28	1/30
Hachinohe	1/29	1/57
Ikoma fault	1/25	1/21
Nankai trough	1/58	1/32

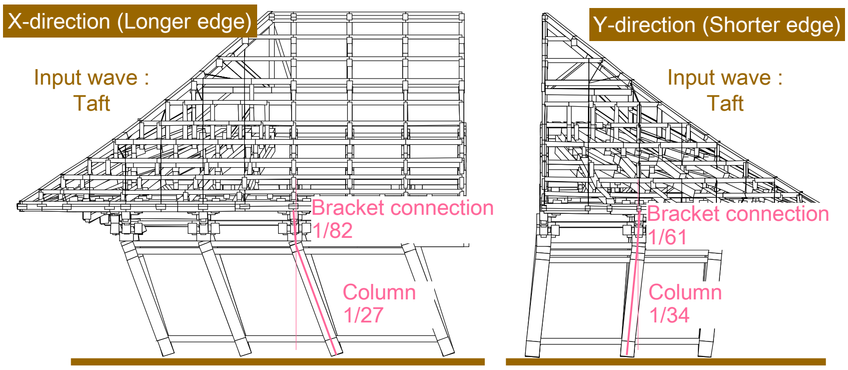


Figure 8 Response for input ground motion ‘Taft’

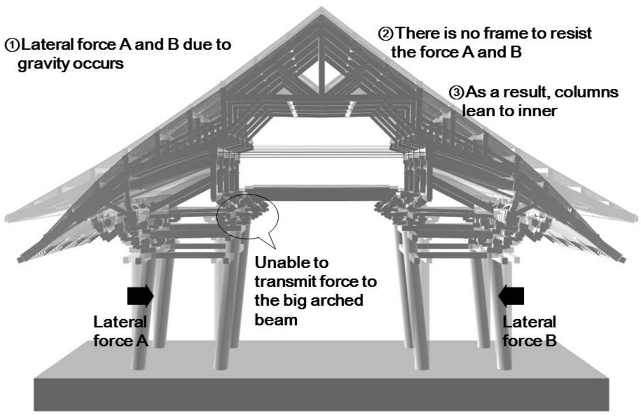


Figure 9 Mechanism of column leaning

RETROFITTING

Fig.10 shows all retrofitting methods considered the beginning. The 1st[t] plan was reinforcing big arched beam located in center. The 2nd plan was reinforcing rafter and its support. The 3rd plan was reinforcing rafter. The 4th plan was reinforcing attic truss. As mentioned in introduction, removable and invisible reinforcements were preferred. Considering how reasonable and how easy to remove, the 4th plan was adopted.

Fig.11 shows force transmissions in the reinforced structure. Vertical force from the center was transmitted to the inner columns from inner side while the attic truss resisting to lateral force from outer side. Moreover, considering the balance between the inertia force and the lateral stiffness of columns, the roof mass was reduced from 320,000 kg to

260,000 kg by reducing roof soil from the top side of the roof. The attic truss reinforcement decreased the leaning rotation to 1/10, and reducing roof mass decreased the leaning to 1/2. In combination, 1/20 leaning rotation was realized.

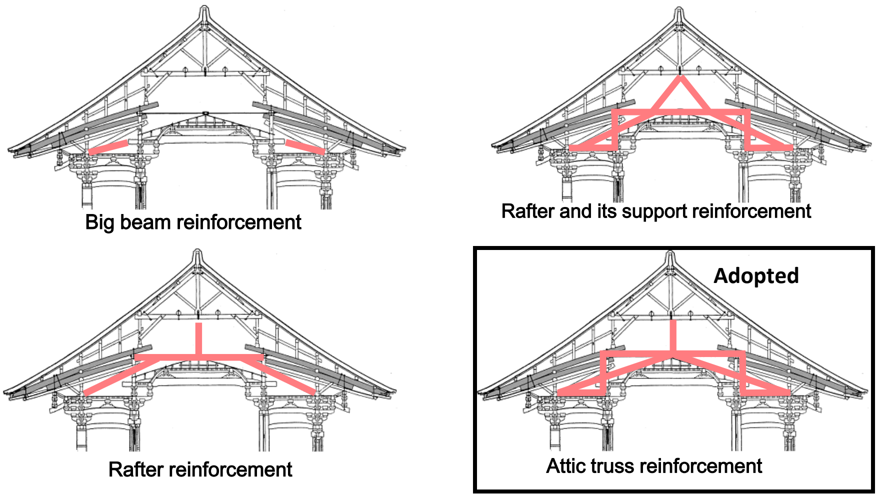


Figure 10 Retrofitting plans

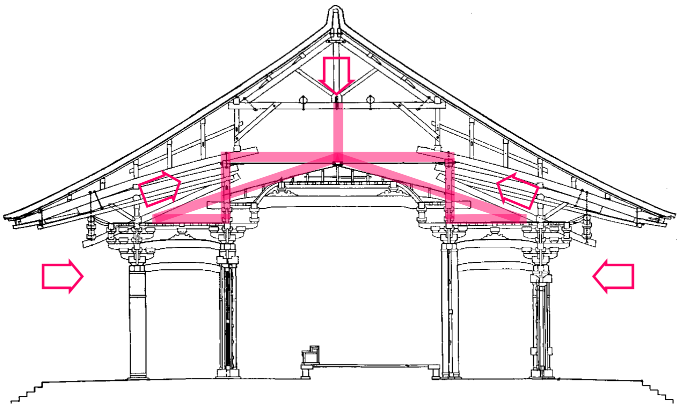


Figure 11 Force transmission in reinforced structure

CONCLUSIONS

Japanese traditional architecture “Toshodaiji-temple” needed retrofitting to correct column leaning and replacing aging members. A Japanese general constructor, “Takenaka corporation” was assigned to be in charge of retrofitting and seismic assessment. Special characteristics of the traditional wood structure were examined through several experiments. The unique and remarkable characteristics such as material property of wood, foundation stone, halving joint, penetrating tie beam, and self-righting column were introduced. In addition to experimental tests, analysis model considering the unique characteristics was built. The behaviors of each components and materials were verified with component tests. With this analysis model, earthquake response analysis and gravity load analysis were conducted. From earthquake response analysis, it was found that the traditional structure was flexible with the maximum response in all input ground motions as about 4% in story drift before retrofitting. From the gravity load analysis, the mechanism of column leaning was examined, and the most appropriate method to retrofit was selected. The adopted retrofitting plans were attic truss reinforcement and the reduction of roof mass. This plan enabled to reduce the column leaning to 1/20 in rotation.

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Title:
Lumped damage mechanics as an alternative to analyse masonry arches

LUMPED DAMAGE MECHANICS AS AN ALTERNATIVE TO ANALYSE MASONRY ARCHES

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ABSTRACT

The preservation of historical constructions means the protection of the cultural heritage of the human civilisation. Therefore, it is necessary to understand the structural behaviour of those constructions. However, generally there is lack of information about such structures since several of those constructions were built a long time ago. Nevertheless, usually the engineers use analytical and finite element analyses to describe the structural behaviour of those structures. Analytical procedures are usually used for simple geometries, because the generalisation of such theory is impractical. Finite element analysis coupled with a nonlinear procedure is preferable, but there are some numerical issues that demand from the engineer a cutting-edge knowledge about such numerical procedure. On the other hand, simplified techniques as lumped damage mechanics might describe satisfactorily the structural behaviour of historical constructions. Such theory is based on well-known concepts of the theory of structures and the inelastic phenomena are lumped on hinges. In the light of the foregoing, this paper presents the lumped damage mechanics as an alternative to analyse historical constructions. For that, unreinforced masonry arches were numerically modelled and compared with experimental observations.

KEYWORDS

Historical constructions; lumped damage mechanics; unreinforced masonry arch; inelastic hinges.

INTRODUCTION

Historical constructions represent a part of the cultural heritage and help to tell the history of human civilisation. There is a concern about the preservation of this cultural heritage, because such constructions were built hundreds or even thousands of years ago. Then, analyse the current and actual situation of historical constructions is an attempt to preserve this heritage. However, gather important information about historical constructions is usually quite difficult. For example, Lourenço (2001) discuss and enumerate some of those difficulties as: lack of geometry information, large variability of mechanical properties, non-applicability of regulations and codes, among others.

Sometimes an intervention planning might be useful for conservation and restoration of monuments and historical constructions. Therefore, the previous knowledge of the structural behaviour is mandatory. For that, theoretical and numerical analyses are usually used (for a review, see Lourenço (2001) and the references therein). Theoretical approaches are very interesting for simple geometries, however for more complex monuments the formulation of such approaches seems to be impractical. Another possibility that appears to be the most used is the finite element analysis, where complex geometries can be easily modelled and nonlinear procedures are adopted e.g. cohesive models, plasticity models, fracture mechanics and continuum damage mechanics. The use of those features requires a vast knowledge in nonlinear analysis and demands experience from the engineer concerning numerical issues that might occur.

Alternatively, simplified approaches can be useful for decision making and preliminary analysis. Among those approaches, the lumped damage mechanics (LDM) presents itself as a powerful tool for analyse complex geometries and it is based on well-known concepts of the theory of structures. This procedure is vastly used to analyse reinforced concrete structures (e.g. Cipollina et al., 1995; Perdomo et al., 1999; Liu and Liu, 2004; Araújo and Proença, 2008; Faleiro et al., 2010; Santoro and Kunnath, 2013), although other materials as steel (Guerrero et al., 2009) and unreinforced concrete (Amorim et al., 2014) were also successfully modelled. In fact, Amorim et al. (2014) also presented a numerical analysis of an unreinforced masonry arch tested by Basilio (2007). In the light of the foregoing, the main objective of this paper is to present LDM as an alternative for analyse historical constructions.

This paper is organised as follows: firstly LDM is presented for circular arches, where the theory of structures is written by using the notation proposed by Powell (1969) and the modelling of inelastic phenomena is made by using an especial logarithm (Amorim et al., 2014; Corless et al., 1996). Thereby, two unreinforced masonry arches (Basilio, 2007; Cancelliere et al., 2010) were analysed showing that the presented procedure can be effectively employed to analyse historical constructions. Finally, some concluding remarks are made and a research frontier for future works is proposed.

LUMPED DAMAGE MECHANICS FOR ARCHES

Statics and kinematics of circular arches

Powell (1969) proposed to rewrite the relations of the theory of structures in a way that the system solution presents information of displacements and element deformations and stresses instead of only nodal displacements, what is obtained in the classical approach. Flórez-López and Proença (2013) originally employed such formulation to circular arched frames. Consider an arch structure composed by *m* members connected to *n* nodes, as the one depicted in Figure 1(a). Each node presents three generalised displacements: two translations in the directions of the global reference axes (X_G and Z_G) and a rotation in the plan X_GZ_G, and three generalised forces: two forces in the direction of the global reference and a bending moment (see Figure 1(a)). Then, the matrices of generalised displacements and nodal forces of the structure are defined as: $\{\mathbf{U}\} = \{u_1 \quad w_1 \quad \theta_1 \quad u_2 \quad \ldots \quad w_n \quad \theta_n\}^T$ and $\{\mathbf{P}\} = \{P_{u1} \quad P_{w1} \quad P_{\theta 1} \quad P_{u2} \quad \ldots \quad P_{wn} \quad P_{\theta n}\}^T$ where the superscript *T* means 'transposed'. Now, consider a circular arch element *b* between nodes *i* and *j*, as illustrated in Figure 1(b). Each element is defined in a local reference system x_bz_b, and such element is characterised by a radius R_b, an arch angle α_b and the angle β_b between the z_b- and Z_G-axes.

The matrix of internal forces of the arch element *b* is $\{\mathbf{Q}\}_b = \{Q_{ui} \quad Q_{wi} \quad Q_{\theta i} \quad Q_{uj} \quad Q_{wj} \quad Q_{\theta j}\}^T$ (see Figure 2(a)) and the generalised stresses of such element are two bending moments at the edges *i* and *j*, and one axial force on the edge *i* (Figure 2(b)), being $\{\mathbf{M}\}_b = \{m_i \quad m_j \quad n_i\}^T$ the matrix of generalised stresses. The matrix of internal forces can be obtained through the matrix of generalised stresses (Flórez-López and Proença, 2013):

$$\{\mathbf{Q}\}_b = [\mathbf{B}]_b^T \{\mathbf{M}\}_b \tag{1}$$

where $[\mathbf{B}]_b$ is a transformation matrix, expressed as:

$$[\mathbf{B}]_b = \begin{bmatrix} 0 & -\dfrac{1}{R_b \sin(\alpha_b)} & 1 & -\dfrac{1}{R_b} & \dfrac{\cos(\alpha_b)}{R_b \sin(\alpha_b)} & 0 \\ 0 & -\dfrac{1}{R_b \sin(\alpha_b)} & 0 & -\dfrac{1}{R_b} & \dfrac{\cos(\alpha_b)}{R_b \sin(\alpha_b)} & 1 \\ 1 & \dfrac{-1+\cos(\alpha_b)}{\sin(\alpha_b)} & 0 & -1 & \dfrac{-1+\cos(\alpha_b)}{\sin(\alpha_b)} & 0 \end{bmatrix} \begin{bmatrix} \cos(\beta_b) & -\sin(\beta_b) & 0 & 0 & 0 & 0 \\ \sin(\beta_b) & \cos(\beta_b) & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & \cos(\alpha_b+\beta_b) & -\sin(\alpha_b+\beta_b) & 0 \\ 0 & 0 & 0 & \sin(\alpha_b+\beta_b) & \cos(\alpha_b+\beta_b) & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \tag{2}$$

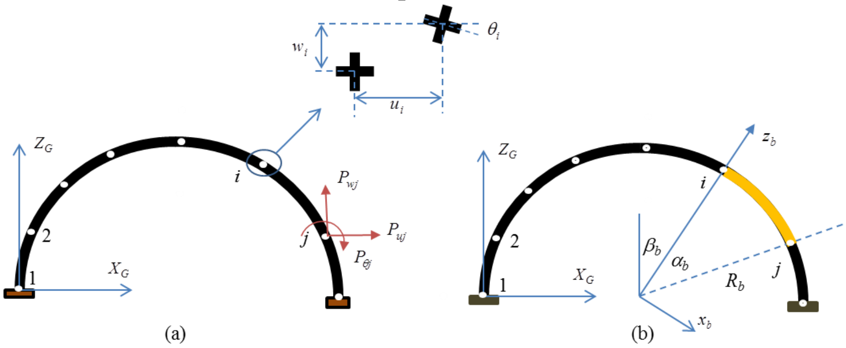


Figure 1 Arch structure: (a) generalised displacements and nodal forces; (b) arched frame element.

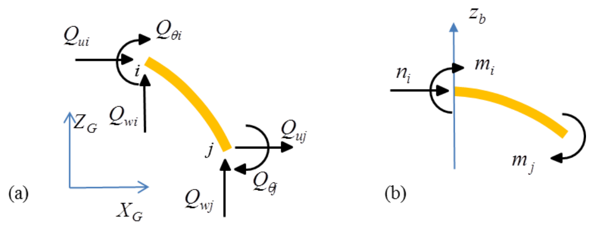


Figure 2 (a) Internal forces in the global reference and (b) generalised stresses.

Thereby, the equilibrium relation can be written as:

$$\sum_{b=1}^{m} \{\mathbf{Q}_E\}_b = \{\mathbf{P}\} \tag{3}$$

where $\{\mathbf{Q}_E\}_b$ is the expanded form of the matrix of internal forces, obtained by the addition of zeros in the positions related to nodes that do not belong to the element:

$$\{\mathbf{Q}_E\}_b = \{\underbrace{0 \quad 0 \quad 0}_{\text{node } 1} \quad \ldots \quad \underbrace{Q_{ui} \quad Q_{wi} \quad Q_{\theta i}}_{\text{node } i} \quad \underbrace{Q_{uj} \quad Q_{wj} \quad Q_{\theta j}}_{\text{node } j} \quad \ldots \quad \underbrace{0 \quad 0 \quad 0}_{\text{node } n}\}^T \tag{4}$$

The mechanical power $\dot{\Lambda}_b$ of the element b can be expressed in terms of the generalised stresses or in terms of the internal forces. For that, a kinematic variable conjugated with the matrix of generalised stresses, denoted matrix of generalised deformations $\{\mathbf{\Phi}\}_b$, is introduced and defined as follows:

$$\dot{\Lambda}_b = \{\dot{\mathbf{\Phi}}\}_b^T \{\mathbf{M}\}_b = \{\dot{\mathbf{U}}\}^T \{\mathbf{Q}_E\}_b \qquad \forall \{\mathbf{M}\}_b \tag{5}$$

The kinematic equation that relates generalised deformations and generalised displacements is obtained by substituting (4) in (5):

$$\{\dot{\mathbf{\Phi}}\}_b^T \{\mathbf{M}\}_b = \{\dot{\mathbf{U}}\}^T [\mathbf{B}_E]_b^T \{\mathbf{M}\}_b \qquad \forall \{\mathbf{M}\}_b; \qquad \text{i.e.} \qquad \{\dot{\mathbf{\Phi}}\}_b = [\mathbf{B}_E]_b \{\dot{\mathbf{U}}\} \tag{6}$$

where the transformation matrix $[\mathbf{B}_E]_b$ is expanded in a similar way as $\{\mathbf{Q}_E\}_b$. Considering that the nonlinear geometric effects are negligible, the transformation matrix can be considered as constant and the final relation in (6) can be written as:

$$\{\mathbf{\Phi}\}_b = [\mathbf{B}_E]_b \{\mathbf{U}\} \tag{7}$$

Constitutive relations of circular arches

The main simplification of this procedure is the assumption that the inelastic effects can be lumped on hinges. Therefore, each element is understood as an assemblage of one elastic arch with two inelastic hinges (Figure 3(a)). Regarding historical constructions as unreinforced masonry arches, in this work it is assumed that the inelastic phenomenon is the cracking of mortar joints (Figure 3(b)). Here, such phenomenon is described by a damage variable $\{\mathbf{D}\}_b = \{d_i \quad d_j\}^T$ that lies between zero and one (Amorim et al., 2014). Then, the matrix of generalised deformations can be understood as a sum between the elastic part $\{\mathbf{\Phi}^e\}_b$ and the part due to damage $\{\mathbf{\Phi}^d\}_b$ i.e. $\{\mathbf{\Phi}\}_b = \{\mathbf{\Phi}^e\}_b + \{\mathbf{\Phi}^d\}_b$. The elastic deformations can be expressed as $\{\mathbf{\Phi}^e\}_b = [\mathbf{F}_0]_b \{\mathbf{M}\}_b$ where $[\mathbf{F}_0]_b$ is the elastic flexibility matrix, which can be obtained by the Castigliano's theorem (Flórez-López and Proença, 2013). For that, the strain energy U_b of an arch element b is expressed by:

$$U_b = \int_0^{\alpha_b} \left(\frac{M(\theta)^2}{2EI_b} + \frac{N(\theta)^2}{2AE_b} \right) R_b d\theta \tag{8}$$

where EI_b and AE_b are the bending and axial stiffnesses, $M(\theta)$ and $N(\theta)$ are the bending moment and axial force distributions over the arch element b (Figure 3(c)). Then, the components of the elastic flexibility matrix are obtained by:

$$F_{11}^0 = \frac{\partial^2 U_b}{\partial m_i \partial m_i}; \quad F_{12}^0 = \frac{\partial^2 U_b}{\partial m_i \partial m_j}; \quad F_{13}^0 = \frac{\partial^2 U_b}{\partial m_i \partial n_i}; \quad F_{22}^0 = \frac{\partial^2 U_b}{\partial m_j \partial m_j}; \quad F_{23}^0 = \frac{\partial^2 U_b}{\partial m_j \partial n_i}; \quad F_{33}^0 = \frac{\partial^2 U_b}{\partial n_i \partial n_i} \tag{9}$$

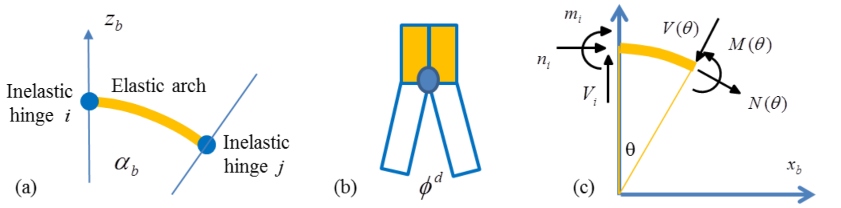


Figure 3 (a) Lumped damage model for an arch element, (b) inelastic rotation due to damage and (c) equilibrium of a generic section of the element.

Now, according to the hypothesis of strain equivalence of the continuum damage mechanics (Lemaitre and Chaboche, 1990), the deformations due to damage can be expressed as $\{\mathbf{\Phi}^d\}_b = [\mathbf{C}(\mathbf{D})]_b\{\mathbf{M}\}_b$ (Perdomo et al., 1999) where:

$$[\mathbf{C}(\mathbf{D})]_b = \begin{bmatrix} \dfrac{d_i F_{11}^0}{(1-d_i)} & 0 & 0 \\ 0 & \dfrac{d_j F_{22}^0}{(1-d_j)} & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{10}$$

Then, the constitutive relation for a circular arch element is expressed as:

$$\{\mathbf{\Phi}\}_b = [\mathbf{F}(\mathbf{D})]_b\{\mathbf{M}\}_b \qquad \therefore [\mathbf{F}(\mathbf{D})]_b = [\mathbf{F}_0]_b + [\mathbf{C}(\mathbf{D})]_b \tag{11}$$

Finally, the damage evolution laws for hinges i and j of the element b are:

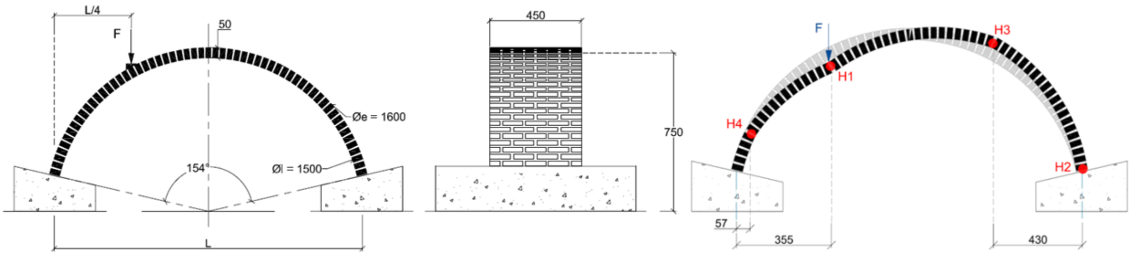
$$\begin{gathered} \begin{cases} \Delta d_i > 0 \Rightarrow G_i = G(d_i) \\ G_i < G(d_i) \Rightarrow \Delta d_i = 0 \end{cases} \therefore \quad G_i = \frac{m_i^2 F_{11}^0}{2(1-d_i)^2}; \quad G(d_i) = \frac{M_{cr}^2 F_{11}^0}{2} \mathrm{W}\left(\frac{q_{un}}{1-d_i}\right)^2 \mathrm{W}(q_{un})^{-2} \\ \begin{cases} \Delta d_j > 0 \Rightarrow G_j = G(d_j) \\ G_j < G(d_j) \Rightarrow \Delta d_j = 0 \end{cases} \therefore \quad G_j = \frac{m_j^2 F_{22}^0}{2(1-d_j)^2}; \quad G(d_j) = \frac{M_{cr}^2 F_{22}^0}{2} \mathrm{W}\left(\frac{q_{un}}{1-d_j}\right)^2 \mathrm{W}(q_{un})^{-2} \end{gathered} \tag{12}$$

where G_i and G_j are the energy release rates of the hinges (Amorim et al., 2014), M_{cr} is the cracking moment, $G(d_i)$ and $G(d_j)$ are the cracking resistant functions of the hinges, $\mathrm{W}(\cdot)$ is the Lambert function (see Corless et al. (1996) for a mathematical discussion) and q_{un} is a parameter for bituminous materials (see Amorim et al. (2014) for a parametric discussion).

UNREINFORCED MASONRY ARCHES

Basilio (2007)

Basilio (2007) tested two unreinforced masonry arches with approximately 1462 mm of span (L) (Figure 4(a)). At failure, both arches presented a classic four-hinge mechanism as depicted in Figure 4(b). Such hinges nucleated sequentially (H1-H2-H3-H4). For the numerical analysis, nodes were positioned where the bending moment distribution presented maximum absolute values, leading to a mesh with four elements (Figure 4(b)). Notice that this arch presents a brittle behaviour (Figure 5), such characteristic can be numerically achieved by excluding the Lambert term of the cracking resistant function or by considering an arbitrarily large value for q_{un}. The second alternative was chosen for the numerical analysis because the exclusion of the Lambert term can present numerical instabilities and no sequential hinge formation is obtained. Then, Figure 5 shows the graph of force vs. displacement for the experimental (Basilio, 2007) and numerical responses. The parameters used in the numerical analysis (EI = 0.75e7 kN.mm², M_{cr} = 150 kN.mm and AE = 36000 kN) were obtained by the material characterisation presented by Basilio (2007).



(a) (b)

Figure 4 Unreinforced masonry arch (Basilio, 2007): (a) geometry (dimensions in mm) and (b) failure mechanism.

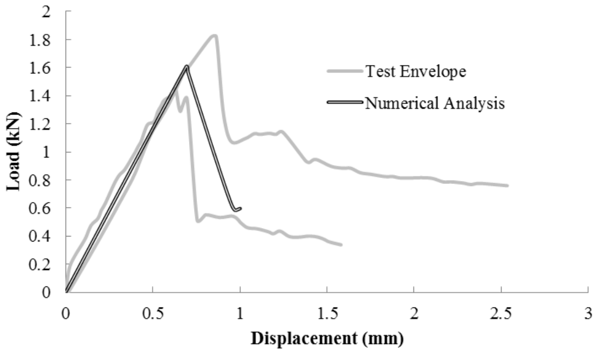


Figure 5 Comparison among experimental (Basilio, 2007) and numerical responses.

Notice that the numerical response is satisfactorily fitted to the experimental envelope (Basilio, 2007). In this analysis, the sequential hinge formation was not completed. The formation of the first hinge (Figure 6(a)) occurred in the peak of the numerical response; and the model lost numerical stability after the formation of the second hinge (Figure 6(b)).

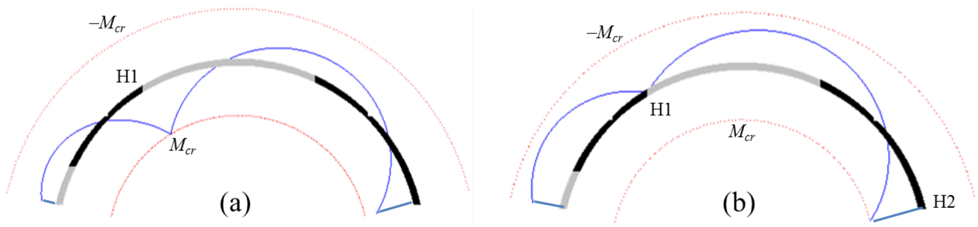


Figure 6 Bending moment diagrams.

Cancelliere et al. (2010)

Cancelliere et al. (2010) carried out experimental tests on two unreinforced masonry arches subjected to monotonic vertical load (Figure 6(a)). As the previous example, this masonry arch presents a sequential hinge formation as the collapse mechanism. Each arch tested by Cancelliere et al. (2010) presented the same collapse mechanism, although some hinges did not appear in the same position (Figure 6(b)). The first hinge (H1) appeared below the applied load, such position was common for both experimental arches; the second hinge (H2) appeared in the joint between bricks #1 and #2 and also was a common position; the third hinge (H3) appeared between bricks #8 and #9 for the specimen called arch I and between bricks #7 and #8 for the arch II; finally, for the arch I the fourth hinge nucleated between bricks #18 and #19 and for the arch II such hinge appeared in the joint between bricks #19 and #20.

Notice that the bricks #1 and #23 do not follow the circular geometry of the arch. Then, those bricks were considered as supports for the numerical analysis. In other words, for the numerical analysis the structure is composed from bricks #2 to #22. Nodes were positioned analogously to the previous example, also leading to a mesh with four elements (elements among joints: #1#2, #7#8, #13#14, #18#19, #22#23). The model parameters (M_{cr} = 23696.64 N.mm, EI = 0.113832e12 N.mm², AE = 0.9486e8 N and q_{un} = 0.000026) were calculated by means of the material characterisation presented by Cancelliere et al. (2010). The comparison among experimental observations (Cancelliere et al., 2010) and numerical response is depicted in Figure 7(a), the four-hinge mechanism numerically obtained is presented in Figure 6(b), and Figure 7(b) shows the damage evolution for each hinge of the collapse mechanism. Figure 8 presents the final configurations of the experimental (Figures 8(a) and 8(b)) and numerical (Figure 8(c)) arches.

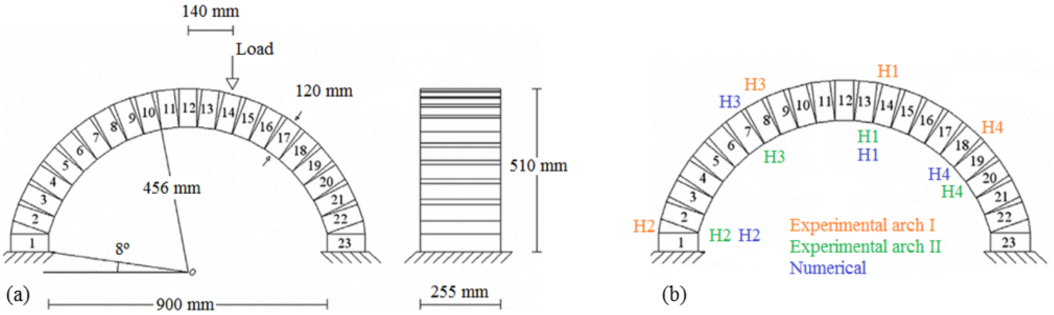


Figure 6 Unreinforced masonry arch: (a) geometry (Cancelliere et al., 2010) and (b) sequence of hinge formation of experimental specimens (Cancelliere et al., 2010) and numerical analysis (this paper).

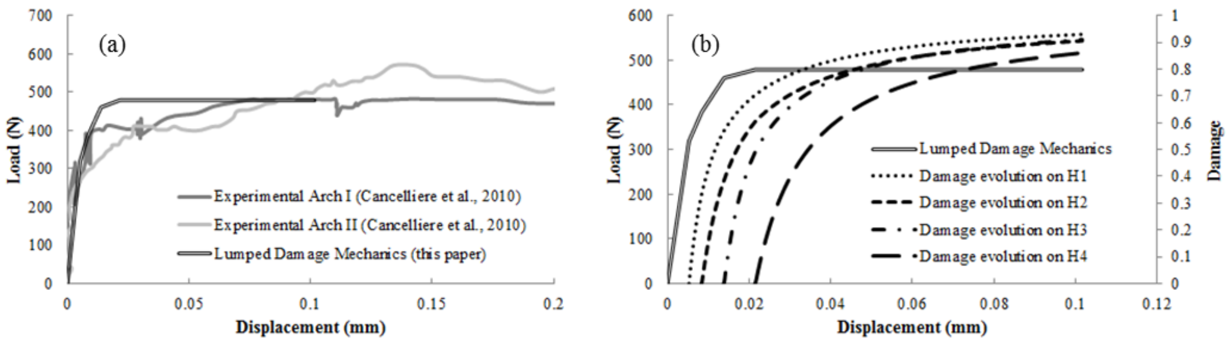


Figure 7 (a) Load-displacement responses and (b) damage evolution.

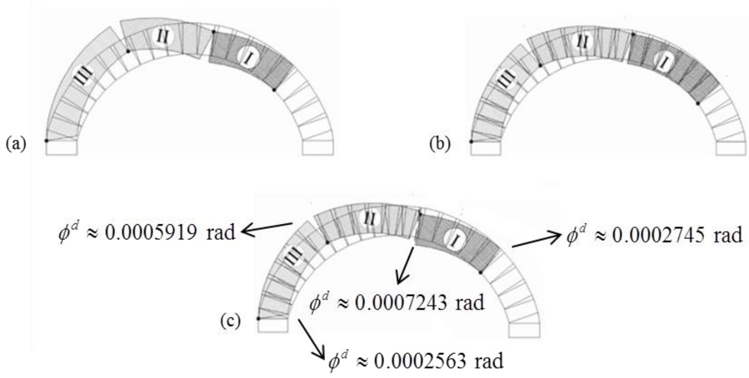


Figure 8 Collapse mechanism of experimental arches (a) I and (b) II (Cancelliere et al., 2010), and (c) numerical analysis (this paper).

The numerical load-displacement response (Figure 7(a)) fits satisfactorily to the experimental results. Regarding the numerical analysis, after the formation of the hinges H1 and H2 the load continue to increase at high rates. Such rate decreases after the formation of the hinge H3 and the load reaches the maximum value while hinge H4 nucleates. Once the collapse mechanism is completed, the load starts a slow decrease until the loss of numerical stability. At that point, the damage variables on the hinges are about 0.93 on H1 (highest value) and 0.86 on H4 (lowest value), characterising the collapse by the formation of a mechanism (Figure 8(c)).

CONCLUSIONS

This paper presented numerical simulations of unreinforced masonry arches by lumped damage mechanics. Once this technique is based on clear and well-known concepts and it is easy to implement and incorporate to finite element programmes, it should be used by engineers and researchers for analyse historical constructions. The model was able to capture different structural force-displacement responses only by the modification of the material parameter. It is noteworthy that such parameter does not have an obvious physical meaning and further studies are needed, as discussed by Amorim et al. (2014).

As aforementioned, LDM presents itself as an interesting tool for analyse structures made of reinforced concrete, steel and plain concrete. Now, it was shown that unreinforced masonry arches can also be modelled. Then, for future works a very first step should be the modelling of other inelastic phenomena (e.g. slipping) for unreinforced masonry arches. After that, the analysis of masonry arches reinforced with fibres can be an interesting application of LDM for analyse restored masonry arches (Cancelliere et al. (2010) restored the collapsed arches with fibres and carried out more experimental tests).

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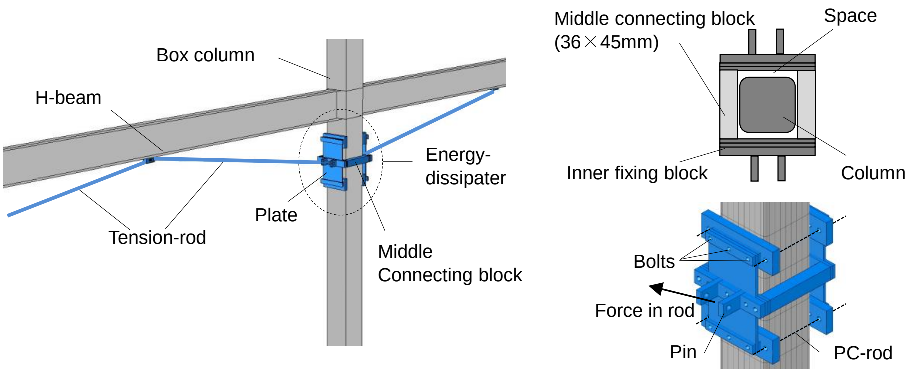
Title:
Minimal-disturbance seismic rehabilitation techniques using tension-only system

Abstract:
Existing building under earthquake loading may suffer serious threat. In order to enhance the seismic performance of buildings, many rehabilitation techniques have been developed to improve seismic resistant capacity. However, current seismic rehabilitation techniques involve large construction, interrupt business and require temporal relocation. In addition, installation includes the use of heavy equipment and arduous work (e.g. welding and gas-cutting). Hence, an innovative rehabilitation technique of steel beam-column connections was developed, that is, minimal-disturbance seismic rehabilitation (MDAD). In the proposed rehabilitation system [shown in Figure 1], a more cost-effective approach was used to not only increase global capacity, but also upgrade local capacity, such as ductility, strength and stiffness. Moreover, since the system contains light-weight steel members without welding and heavy construction equipment, it is also considered as a convenient rehabilitation technique. To better illustrate the utility of MDAD and to capture the benefit, a four-story two-bay steel moment resisting frame was studied with OpenSees. The performance of the steel moment resisting frame with MDAD was evaluated in static and dynamic analyses.



(a) (b)

Figure 1 Schematic of rehabilitation system: (a) overall shape; (b) attachment to existing frame.

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Ikumi Hamashima is an undergraduate student at the school of Architecture, Kyoto University. Ms. Hamashima graduated from Nishiyamato gakuen High school and entered Kyoto University in 2011. Now she is studying hard for her thesis. Her research topic is about the sliding behavior of free-standing structures with the application of graphite lubricant to column bases. Her research advisors are Prof. Nakashima and Prof. Kurata. She belongs to seismic rehabilitation group with Dr. Hu, Mr. Zhang and Ms. Sato. She likes sleeping and eating. Her favorite sweet is chocolate. Without chocolate, she cannot live.

Title:
Experimental evaluation of free-standing steel structures with carbon powder between column bases and foundation

Abstract:
Free-standing structure is a structure with its column bases detached from foundation. The seismic damage under earthquake loadings are reduced by capping the shear force transmitted to the upper structures with controlled friction. Such system is not new as traditional Japanese wood buildings such as shrines and temples often adopt the sliding mechanism between columns and foundation stones.

The key parameter that controls the behavior of free-standing structures is the friction between column bases and foundation. Past works reported that the friction between steel column bases and mortar was stable for repeated loadings but the friction coefficient was as high as 0.8. However, by using low-cost carbon powder as solid lubricant between column bases and foundation, the friction coefficient was reduced to around 0.2. A series of shaking table tests using simplified building specimen (Figure 1) verified significant reduction in shear force demand to the upper structure with lubricated column bases.

The current focuses of the research are on the investigation for the behaviors under sinusoidal waves and earthquake ground accelerations. Parameters considered in numerical analysis and shake table tests include the mass ratio between base and upper structure, biaxial and vertical loadings. The influence of the variations in friction coefficient on rotational deformation and the rocking behavior under relatively large overturning moments are also studied.

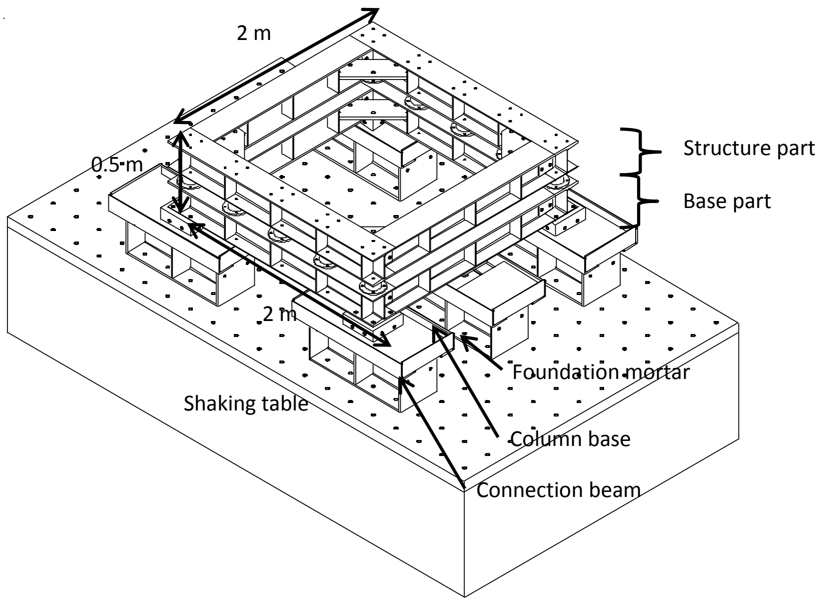

Figure 1 Specimen of free-standing structure.

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Liusheng HE received his B.S. from Dalian University of Technology, China, in 2008, and M.S. from Tongji University, China, in 2011. He is currently working towards his Ph.D. at DPRI of Kyoto University, pursuing research in steel slit wall dampers with embedded structural condition assessment capability.

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Takuma TOGO graduated from Kyoto Institute of Technology and he is currently working towards his master degree at Kyoto University. His research interests aresteel slit wall dampers capable of condition assessment and multi-row slit shear wall using low yield steel.

Title:
Steel shear walls with double-tapered links capable of condition assessment

Abstract:
A growing attention has been paid to structural health monitoring (SHM) systems where sensors deployed to structures provide objective data for assisting the assessment of structural conditions after extreme events. However, SHM systems require large initial investments on senor installations and their maintenance fee for continuous operations. Alternatively, if energy dissipation devices commonly used in buildings have capability in memorizing their maximum experienced deformation, these devices can work as indicators for the extent of building damage.

The concept of using a hysteretic damper as a condition assessment device that functions immediately after a damaging earthquake is realized by making use of the residual out-of-plane deformation of links that are arranged in slit shear walls. According to the proposed inspection procedure, the maximum drift ratio experienced by the slit wall is estimated based on the number of torsionally deformed links whose dimensions are determined so that the links would exhibit notable torsional deformation at the target deformations. The adoption of a double-tapered shape for the links enables to significantly increase the amount of out-of-plane deformation. The relationship between the dimensions and the torsional deformation of the links is established using numerical simulations. The effectiveness of the proposed condition assessment scenario is verified by using a series of cyclic-loading tests for individual links and groups of links.

Steel shear walls with double-tapered links capable of condition assessment

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Introduction

Steel slit shear walls are commonly installed in buildings as the lateral resistant system to reduce the seismic response. Meanwhile, structural condition assessment is increasingly needed, especially right after a major earthquake event. The work here attempts to add the condition assessment by introducing double-tapered links into the steel slit shear walls, as a result to achieve a wider applicability for new steel slit shear walls. The condition assessment capability is obtained by the visual inspection of out-of-plane torsional deformation of double-tapered links.

Experimental design

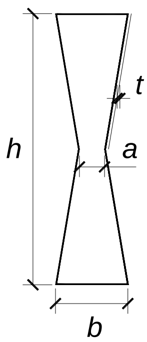
The double-tapered shape is adopted to relocate the region with large plasticity from link ends towards the middle part due to the tapering shape. A schematic of a double-tapered link is illustrated in Fig. 1, where *b*, *a*, *h* and *t* denote the link end-section width, mid-section width, height, and thickness, respectively. The taper with end-section and mid-section width ratio of 3 (b/a=3) is selected to have large plasticity occurred at quarter-height sections.

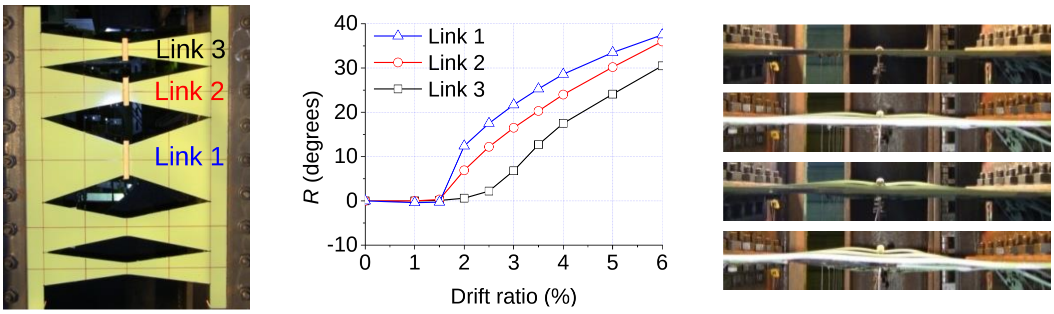
Under in-plane shear deformation, the links yield and buckle out of plane involving torsional deformation. Compared with the rectangular links, the torsional deformation is initiated earlier and grows larger. Parametric study in simulation shows that the torsional deformation was mainly controlled by the width-to-thickness ratio $\lambda = 2a/t$ while little affected by the aspect ratio $\beta = h/2a$ as long as the link is neither too long nor too short.

Ten pairs of steel slit shear walls were designed. Width-to-thickness ratio and aspect ratio were the two major parameters studied, with λ ranging from 9 to 23 and β ranging from 3 to 10. Figure 2(a) shows the detail of one specimen.

Test results

Experimental results verified the little influence of the aspect ratio and controlling effect of the width-to-thickness ratio. Wider links buckled earlier, followed by the buckling and succeeding torsional deformation of narrower links (Fig. 2(b)). Through the inspection of notable torsional deformation of links with different widths occurred at specific drifts, structural condition in terms of the experienced maximum shear deformation was estimated (Fig. 2(c)).





(a) (b) (c)

Fig. 1 Schematic of a double-tapered link.

Fig. 2 One tested specimen: (a) before loading; (b) progress of torsional deformation in terms of the rotation (*R*) of the mid-section; (c) down-top views at drift ratios of 0, 2%, 2.5% and 3.5%.

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Akiko Suzuki is a 1st-year graduate student of the Department of Architecture and Architectural Engineering at Kyoto University. Ms. Suzuki completed her bachelor studies at Faculty of Engineering Undergraduate School of Architecture at Kyoto University. Her current research interests are in the areas of post-disaster structural damage screening and aftershock risk assessment. Her work includes development of autonomous structural monitoring system using advanced sensing technologies and residual seismic performance evaluation with model updating method.

Title:
Damage assessment of steel beam-column connections using ambient-based inner-force estimates

Abstract:
After the 2011 Tohoku earthquake in Japan, post-earthquake diagnosis tools that quantify the extent of damage in structural components of buildings are particularly needed for coming earthquakes to set up an evacuation and re-occupancy policy based on the actual state of the monitored building structures. At this stage, however, damage in earthquake-affected buildings is visually investigated by structural engineers, and its objective assessment or quantitative evaluation hasn't been investigated yet.

This research presents a novel testing method that identifies local damage extent based on the modal vibratory characteristics of steel beam-column connection using ambient vibration responses. In the proposed test configuration, a specimen of structural components is installed to a resonance frame that supports large fictitious mass, and the resonance frequency of the entire system is set as the natural frequency of a mid-rise steel building. The specimen is damaged quasi-statically, and resonance vibration tests are conducted with modal shaker at the presence of notable damage. The proposed method enables the vibratory evaluation of realistic damage in structural components without constructing a large specimen of entire structural system.

Two kinds of specimens, with floor slab or without, are tested to validate the influence of the loss of composite action induced by cracks in floor slabs and fractures in steel beams for vibratory characteristics. The transition of the neutral axis and the reduction of the root mean square (RMS) of the dynamic strain response were proposed as damage sensitive features to detect the slab damages and beam damages respectively. These experimental results were extended to building model updating which comprehensively accounted for both damage, cracks in floor slab and fracture in steel beam. The analytical fiber model was built with open system for earthquakes engineering simulation software, OpenSees. The damage extent in floor slab were identified by removing floor slab sections corresponding to the objective function of neutral axis, while that in steel beam by reducing the rotational stiffness of damage spring which model fractures at beam ends. The residual seismic performance was estimated in term of stiffness, maximum strength and damage state with the proposed model updating method based inner-force estimates, and the correspondence to the experimental results was shown with a certain error.

DAMAGE ASSESSMENT OF STEEL BEAM-COLUMN CONNECTIONS USING AMBIENT-BASED INNER-FORCE ESTIMATES

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Introduction

After the 2011 Tohoku earthquake in Japan, post-earthquake soundness assessment that quantify the extent of damage in structural components of buildings are particularly in need for coming earthquakes to set up an evacuation and re-occupancy policy based on the actual state of the monitored building structures. Soundness of the earthquake-affected building can be evaluated as the safety margin to its collapse, at this stage, however, its objective assessment or quantitative evaluation hasn't been investigated yet. It's possible to evaluate soundness of damaged buildings if the residual seismic performances, such as the reduction of stiffness and strength, can be quantitatively evaluated based on the local damage identification.

Damage sensitive features vary to structural types and building materials. While cementitious composites change its stiffness properties with yielding, metals keep their Young's modulus until strong geometric nonlinearity occurs. This incidence characterizes the vibratory characteristics of steel buildings with composite floor slabs, comprising reinforced concrete cast, depends on where the damage occurs, that is, in a steel frame or in floor slabs. The influence of the damage in floor slab to the lateral stiffness or modal properties is considerable enough to be equivalent to the bottom flange fracture in steel beam, although the latter contributes the deterioration of the strength much more than the former. That indicates the difficulty to evaluate these damages separately only by the change in the lateral stiffness or modal properties.

Autonomous damage evaluation tool

To develop an autonomous damage evaluation tool for structural components of steel building, this paper presents a novel testing method that evaluates local damage extent based on the modal vibratory characteristics of steel beam-column connection by simultaneously simulating seismic damage and ambient vibration responses (Figure 1, 2). In the proposed test configuration, a specimen of structural components is installed to a resonance frame that supports large fictitious mass and the resonance frequency of the entire system is set as the natural frequency of a building structure. The resonance frame is interfaced with a quasi-static loading system and a modal shaker. The specimen is damaged quasi-statically and, resonance vibration tests are conducted with the modal shaker at the presence of notable damage. The proposed method enables the vibratory evaluation of realistic damage in structural components without constructing a large specimen of entire structural systems. A proof-of-concept test was conducted with a quarter-scale beam-column connection of a mid-rise steel building. The changes in vibratory characteristics were monitored using dynamic strain sensors and accelerometers. The performance of the proposed testing method was first verified and, then

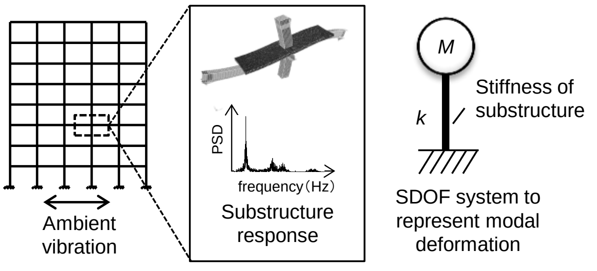

Figure 1. Response under ambient vibration and equivalent SDOF system for substructure.

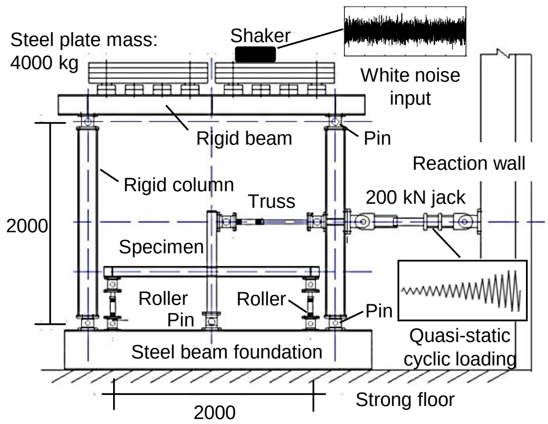

Figure 2 Test setup.

the changes in vibratory characteristics based on inner-force distribution of the specimen were examined. Two kinds of specimens, with floor slab or without, were tested to validate the influence of the loss of composite action induced by cracks in floor slabs and fractures in steel beams for vibratory characteristics. The transition of the neutral axis (Figure 3) and the reduction of the root mean square (RMS) of the dynamic strain response (Figure 4) were proposed as damage sensitive features for the slab damage level and for beam damage level respectively.

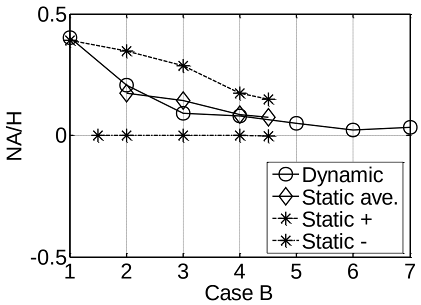

Figure 3 Transition or neutral axis (With slab).

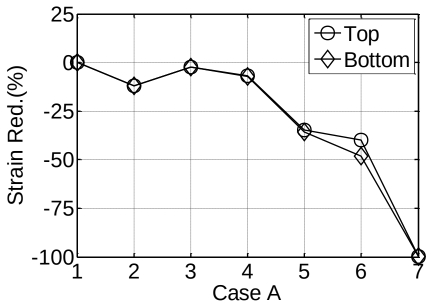

Figure 4 Dynamic strain response (No slab).

Residual Capacity Assessment via Model Updating

Model updating method which comprehensively accounted for both damage, cracks in floor slab and fracture in steel beam was proposed using the experimental results for the two damage sensitive features mentioned above. The analytical fiber model was built with open system for earthquakes engineering simulation software, OpenSees. The damage extent in floor slab were identified by eliminating the floor slab section gradually based on the objective function of neutral axis (Figure 5), while that in steel beam was identified by reducing the rotational stiffness of damage spring which model the fracture at beam end (Figure 6). The residual capacities (lateral stiffness, strength, and reduction of rotational stiffness at fractured section) were estimated, and the correspondence to the experimental results was shown with a certain error.

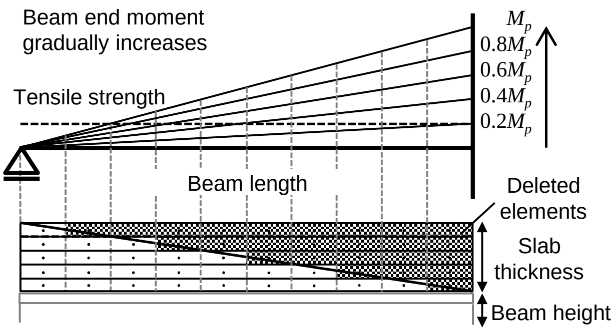

Figure 5 Crack propagation model

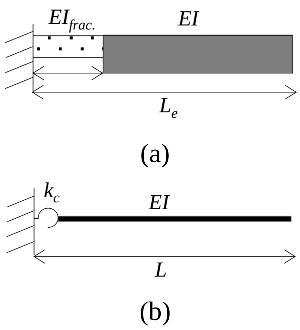

Figure 6 Beam fracture model
(a) Effective length model (b) Spring model

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Title:
Damage detection of beam fractures in steel buildings under earthquake loading.

Abstract:
Delays in the post-earthquake safety estimations of important buildings significantly increase unnecessary disorder in economic and social recovery following devastating earthquakes. Structural health monitoring (SHM) that allows rapid and reliable damage evaluation on buildings is recognized as one of the best means potential to support post-quake decision-making, to optimize crisis management, and thus to mitigate seismic disaster. Providing promptness and objectivity in evaluation procedures, damage detection through a SHM system using sensors attracts attention from building owners and other stakeholders. Nonetheless, local damage on individual structural elements is not easily identifiable, as such damage weakly relates to the global vibrational characteristics of buildings. The primary objectives of this research are to present and verify a method that quantifies the amount of local damage (i.e., fractures near beam-column connections) for the health monitoring of steel moment-resisting frames that have undergone a strong earthquake ground motion. In this research, a novel damage index based on the monitoring of dynamic strain responses of steel beams under ambient vibration before and after earthquakes is firstly presented. The effectiveness of the damage index and an associated wireless strain sensing system are examined with a series of vibration tests using a five-story steel frame test bed.

DAMAGE DETECTION OF BEAM FRACTURES IN STEEL BUILDINGS UNDER EARTHQUAKE LOADING

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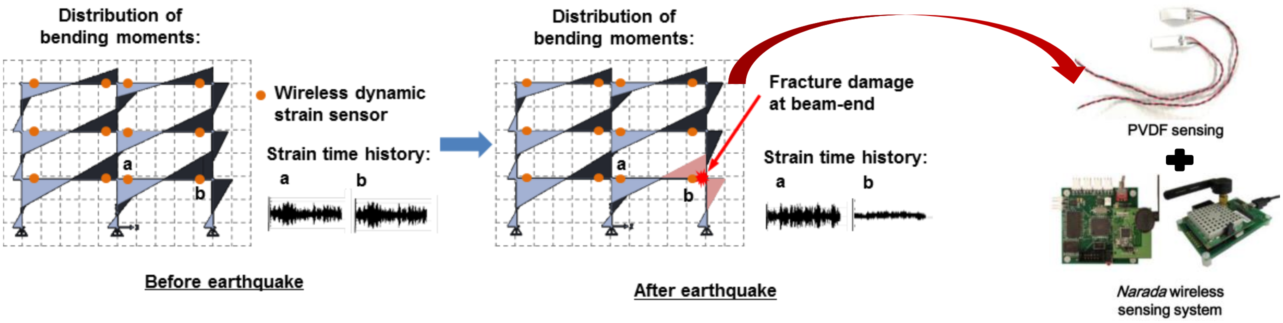
ABSTRACT

Delays in the post-earthquake safety estimations of important buildings significantly increase unnecessary disorder in economic and social recovery following devastating earthquakes. Structural health monitoring (SHM) that allows rapid and reliable damage evaluation on buildings is recognized as one of the best means potential to support post-quake decision-making, to optimize crisis management, and thus to mitigate seismic disaster. Providing promptness and objectivity in evaluation procedures, damage detection through a SHM system using sensors attracts attention from building owners and other stakeholders. Nonetheless, local damage on individual structural elements is not easily identifiable, as such damage weakly relates to the global vibrational characteristics of buildings. The primary objectives of this research are to present and verify a method that quantifies the amount of local damage (i.e., fractures near beam-column connections) for the health monitoring of steel moment-resisting frames that have undergone a strong earthquake ground motion. In this research, a novel damage index based on the monitoring of dynamic strain responses of steel beams under ambient vibration before and after earthquakes is firstly presented. The effectiveness of the damage index and an associated wireless strain sensing system are examined with a series of vibration tests using a five-story steel frame test bed.

Methodology:



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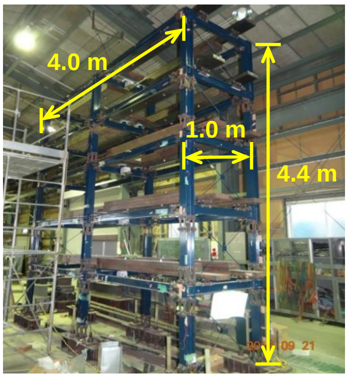
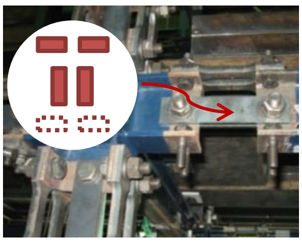
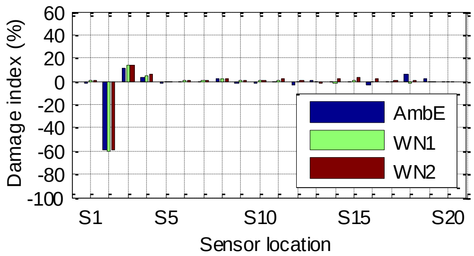


Test bed



Beam fracture



Damage index

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Title:
Preliminary Validation of a Multimodal Adaptive Procedure

PRELIMINARY VALIDATION OF A MULTIMODAL ADAPTIVE PROCEDURE

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ABSTRACT

The nonlinear static method of analysis is allowed by modern seismic codes for seismic assessment of existing buildings and Eurocode 8 suggests the N2 method developed by Fajfar et al. This method evaluates the performance curve of the structure by a pushover analysis and relates each point of this curve to a value of peak ground acceleration (a_g) through the equivalent SDOF system. One of the proposed variants of the original N2 method has been recently developed by Ghersi et al. It is a multimodal and adaptive procedure, which does not require the definition of the equivalent SDOF system and relates directly each point of the performance curve to the corresponding value of a_g. In this paper the procedure is applied to four steel moment resisting frames. Its effectiveness is investigated by comparing the obtained performance curve to that provided by the original N2 method and to the actual seismic response determined by incremental dynamic analysis. The performance curve is represented in terms of base shear (or peak ground acceleration) and top displacement.

KEYWORDS

Nonlinear static method; adaptive procedure; existing structures, performance curve

INTRODUCTION

A proper assessment of existing structures requires a displacement-based approach, which compares the displacement capacity to the displacement demand. The evaluation of the time-history of inelastic response to a given accelerogram can be obtained by means of nonlinear dynamic analysis. Unfortunately, the difficulties in correctly modelling the nonlinear cyclic behaviour of structural members and in properly simulating the seismic input make this kind of analysis not recommended for every day design use. The need for a tool that explicitly considers the inelastic deformation experienced by the structural members during earthquakes without carrying out complex and computational costly nonlinear dynamic analysis led researchers to develop nonlinear static methods, which aim at predicting in a simplified way the seismic response.

In this paper, the N2 method proposed by Fajfar et al. (1989, 1996, 1999) is examined. This method requires the evaluation of the base shear versus top displacement relationship (performance curve) and relates each point of this curve to the value of peak ground acceleration a_g. The base shear versus top displacement relationship is determined by pushover analysis. Then, as an intermediate step, a Single-Degree-Of-Freedom (SDOF) system equivalent to the examined structure is used to relate each point of the performance curve, and thus the displacement demand, to the value of a_g. Here an innovative nonlinear static method of analysis is analysed and hereinafter it will be named "Displacement Adaptive Procedure" (DAP). This procedure, which was proposed by Ghersi et al. (2011, 2013), involves the use of modal analysis with incremental response spectrum. At each step, the modes of vibration are recalculated considering the stiffness matrix corresponding to the current state of damage of the structure. The modal contributions corresponding to a reference value of a_g are combined to obtain a displacement profile that is used as load vector. Then, the reference a_g is scaled up to the attainment of the next plastic hinge. The DAP provides important advantages. In fact, the explicit reference to a SDOF system is not necessary. Moreover, this method takes into account the influence of higher modes of vibration on the structural response and the variation of the dynamic properties of the structure due to the increase of seismic excitation level.

In order to validate the proposed nonlinear static method, it is applied on a set of steel framed structures. The displacement demand is determined for different levels of seismic excitation by DAP, N2 method and incremental dynamic analysis (IDA). Then, the response prediction obtained by DAP is compared to that provided by the N2 method and to the "actual" seismic response obtained by IDA.

THE N2 METHOD

Several proposals of nonlinear static methods of analysis can be found in literature. Among these, the N2 method was adopted by the Italian and European seismic codes (CEN. EN 1998-1, D.M. 2008). It allows the evaluation of the seismic response of the structure in two steps: first, the performance curve of the structure is determined by means of pushover analysis and, then, the displacement demand is obtained through the analysis of an equivalent SDOF system subjected to a seismic input defined by the response spectrum.

The performance curve of the structure, represented in terms of base shear versus top displacement relationship, is evaluated by pushover analysis. The distribution of the horizontal forces F_i along the height of the structure is obtained by multiplying the floor masses m_i by a displacement profile Φ. However, it is recommended that the analysis be repeated by two displacement profiles that bound the actual seismic response of the structure. Generally, seismic codes recommend displacement profiles which are proportional to the first mode of vibration and constant along the height.

The top displacement demand of the actual Multi-Degree-Of-Freedom (MDOF) system corresponding to a given a_g is obtained by: (1) defining a SDOF system equivalent to the structure under examination, (2) calculating the displacement demand of this system and (3) finally converting it into the top displacement demand of the actual MDOF system.

If a modal displacement profile Φ is used, the mass m^* of the equivalent SDOF system is related to the effective modal mass M^* corresponding to the considered mode shape by the equation

$$m^* = \frac{M^*}{\Phi_n \, \Gamma} \tag{1}$$

where Φ_n is the value of displacement at the top storey and Γ is the modal participation factor.

$$\Gamma = \frac{\sum m_i \, \Phi_i}{\sum m_i \, \Phi_i^2} \tag{2}$$

The performance curve of the MDOF system is scaled by means of the following equations:

$$F^* = \frac{V_b}{\Phi_n \, \Gamma} \tag{3a}$$

$$D^* = \frac{D}{\Phi_n \, \Gamma} \tag{3b}$$

and the capacity curve of the equivalent SDOF system is then obtained by idealizing, within the relevant range of displacements, the obtained curve by a bilinear relationship characterised by a lateral strength F_y^* and a yield displacement D_y^*. Different equivalence conditions have been suggested in literature or by codes. In this paper, the post-yield slope is assumed null according to Fajfar (Fajfar *et al*., 1999), F_y^* is equal to the maximum value of F^* within the relevant range of displacements and D_y^* is calculated by equating the areas under the original and the idealised curve. The slope of the elastic branch is equal to the ratio $K_s = F_y^* / D_y^*$ and it is called "secant stiffness". The period of the idealised SDOF system is:

$$T^* = 2\,\pi \sqrt{\frac{m^*}{K_s}} \tag{4}$$

According to Fajfar *et al*. (1999), the displacement demand D_{req}^* of the inelastic SDOF system is related to the spectral value $S_{de}(T^*)$ by the following equations

$$D_{req}^* = S_{de}(T^*), \qquad T^* \geq T_C \tag{5a}$$

$$D_{req}^* = \frac{1}{R_\mu}\left[1 + \left(R_\mu - 1\right)\frac{T_C}{T^*}\right] S_{de}(T^*), \quad T^* < T_C \tag{5b}$$

where T_C is the transition period that separates the constant acceleration branch of the spectrum from the constant velocity branch and R_μ is the force reduction factor (ratio of the elastic strength demand to the actual strength of the bi-linear system).
Finally, the displacement demand of the equivalent SDOF system is transformed back to the top displacement demand of the MDOF system by the inverse of Eq. 3b

$$D_{req} = \Phi_n \, \Gamma \, D^*_{req} \tag{6}$$

The seismic response of the MDOF system, in terms of member internal forces, floor displacement, plastic deformations, etc. is then assumed as that obtained by pushover analysis at a top displacement equal to D_{req}. If any response quantity is larger for a top displacement smaller than D_{req}, this maximum value has to be used instead.

THE PROPOSED PROCEDURE: DISPLACEMENT ADAPTIVE PROCEDURE

This procedure, called Displacement Adaptive Procedure (DAP), is described with reference to a plane frame. The proposed approach involves the use of a multimodal response spectrum analysis with incremental seismic input. The procedure adopts a numerical model based on an ensemble of beam-column members with plastic hinges at their ends.

The nonlinear analysis is carried out on the frame which already sustains gravity loads in seismic combination. A modal response spectrum analysis of the structure is performed at each (i-th) step of the analysis. The structural model is representative of the yielding undergone by the structure in the previous steps. In particular, the structural model is updated at the end of each step by replacing the yielded cross-section with a hinge. The use of hinges is admitted if a rigid-perfectly plastic relation is used to model the nonlinear behaviour of the end cross-sections of the structural members. Otherwise, rotational springs with stiffness equal to the residual one due to hardening has to be used. In this paper, plastic hinges with rigid-perfectly plastic behaviour are adopted.

The elastic response spectrum corresponding to the considered soil type and scaled at a reference value of a_g is used. Here, the reference value of a_g is assumed equal to unity for all the steps of the nonlinear analysis. The modal contributions to the response are determined. Then the maximum storey displacements are evaluated using a combination rule for modal responses. These displacements provide the load vector corresponding to a unitary a_g. This displacement profile is applied to the frame. The corresponding internal forces, for instance the bending moment $M_i^{a_g=1}$, are scaled by the Δa_g, cumulated to those attained at the end of the previous step ($M_{i-1}^{a_{g,i-1}}$) and equated to the yielding value (M_{pl})

$$M_i^{a_g} = M_{i-1}^{a_{g,i-1}} + \Delta a_g \; M_i^{a_g=1} = M_{pl} \tag{7}$$

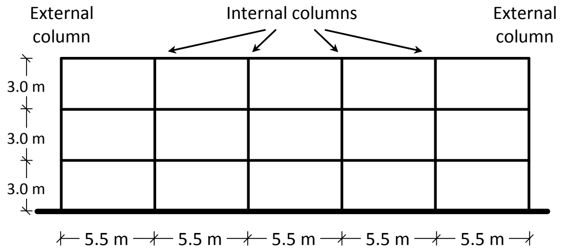
This equation is solved for each end cross-section of all the members and the minimum value of $\Delta \, a_g$ represents the increase of the peak ground acceleration ($\Delta a_{g,i}$) that causes the next plastic hinge. Finally, the a_g corresponding to the end of the current step $a_{g,i}$ is calculated as follows

$$a_{g,i} = a_{g,i-1} + \Delta \, a_{g,i} \tag{8}$$

The Equation(7) is used for all the response parameters assuming $\Delta a_g = \Delta a_{g,i}$ to evaluate the seismic response at the end of the i-th step. Step by step the DAP analysis continues until a predetermined limit state is reached.

VALIDATION OF DAP METHOD

In order to assess the effectiveness of the proposed nonlinear static method of analysis, a set of four steel moment resisting frames has been considered. The validation involves two stages. First, the performance curve of the considered frames determined by the DAP is compared to that obtained by the N2 method. In this paper, the N2 method is applied as specified in Eurocode 8 (EC8). The N2 method is applied considering two load patterns: horizontal forces proportional to the first mode of vibration and proportional to the storey masses. Hereinafter these load patterns are called "first mode" and "constant" load patterns. Second, the performance curve obtained by DAP and N2 method is compared to the actual performance curve of the frames determined by incremental nonlinear dynamic analysis. The performance curves are considered in terms of base shear V_b and top displacement D_t, or peak ground acceleration a_g and D_t. The IDAs and the pushover analyses are carried out by the DRAIN-2DX (Prakash *et al.*, 1993) and the TEL2008 (Ghersi, 2008) computer program, respectively. Specifically, the DAP has been implemented in this program. The peak ground acceleration a_g in IDA increases with step of 0.02 g until the top displacement reaches 4% of the height of the frame. A member-by-member modelling with plastic hinges assigned at member ends is adopted. A Rayleigh viscous damping is used and set at 5% for the first two modes of vibration. Strain hardening and geometrical nonlinearity are not considered. According to EC8, nominal dead loads plus quasi-permanent live loads are assumed as initial gravity loads in the analyses. The elastic spectrum proposed by EC8 for soil type D is used for nonlinear static

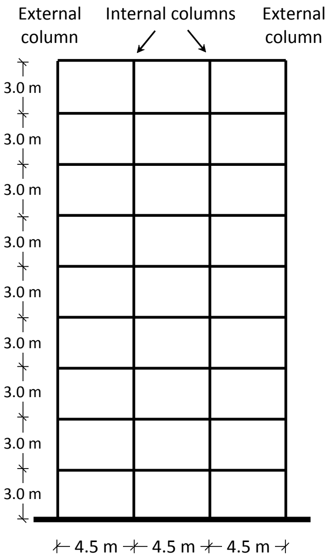


	3ADH		3ADHsm	
Storey	External	Internal	External	Internal
3	HEB 180	HEB 180	HEB 100	HEB 100
2	HEB 200	HEB 200	HEB 100	HEB 100
1	HEB 200	HEB 200	HEB 100	HEB 100

Fig. 1: Geometrical scheme and column cross-sections of the 3ADH and 3ADHsm frames.

methods; a set of ten artificial accelerograms, generated by the SIMQKE computer program and compatible with the assumed elastic response spectrum, is used for incremental nonlinear dynamic analysis.

Description of the analysed frames
A set of four frames is chosen for the numerical analysis in order to include a wide range of structural features, which influence seismic response and its estimation by nonlinear static methods. The frames are different for the type of collapse mechanism, fundamental period, number of storeys and gravity loads. Wide-flange shapes available in Europe and steel grade with yield stress f_y = 275 MPa are used for beams and columns. Two of the four frames are drawn from the set of 108 frames analysed in Bosco et al. (2009) in which they are named 3ADH and 8CFL. The frame 3ADH is 3-storey high, with 5 spans 4,5 m long and IPE220 beams, which sustain gravity loads equal to 5,58 kN/m (Fig. 1). Instead, the frame 8CFL is 9-storey high, with 3 spans 5,5 m long and IPE270 beams, which sustain gravity loads of 20,00 kN/m (Fig. 2). Columns are designed so that the two frames achieve a global collapse mechanism and their cross-sections are presented in Figure 1 and Figure 2. Columns of the frames 3ADH and 8CFL are redesigned to sustain gravity loads only. This second design leads to frames that attain a soft storey collapse mechanism. The obtained frames are named 3ADHsm and 8CFLsm and their column cross-sections are summarised in Figure 1 and Figure 2, respectively. The suffix “sm” after the main code points out that these frames will experience a storey mechanism. The fundamental periods of the four considered frames range from 0,50 s (3ADH) to 2,90 s (8CFLsm).



	8CFL		8CFLsm	
Storey	External	Internal	External	Internal
9	HEB 220	HEB 220	HEB 100	HEB 100
8	HEB 260	HEB 260	HEB 100	HEB 140
7	HEB 280	HEB 280	HEB 120	HEB 160
6	HEB 300	HEB 300	HEB 140	HEB 180
5	HEB 300	HEB 300	HEB 140	HEB 180
4	HEB 300	HEB 300	HEB 160	HEB 200
3	HEB 300	HEB 300	HEB 160	HEB 220
2	HEB 300	HEB 300	HEB 160	HEB 220
1	HEB 300	HEB 300	HEB 180	HEB 240

Fig. 2: Geometrical scheme and column cross-sections of the 8CFL and 8CFLsm frames.

Comparison among DAP, N2 method and IDA analysis

Figure 3 compares the performance curves in terms of base shear V_b and top displacement D_t obtained for the analysed frames by the considered nonlinear static procedures and by IDA. The rhombuses pinpoint the base shear and top displacement demands provided by IDA for the considered values of a_g. The other three curves are obtained by applying the DAP (one curve) and the N2 method (two curves). In particular, the dashed line identifies the performance curve obtained by DAP. Instead, the thick and thin continuous lines represents the results provided by the N2 method with first mode and constant load patterns, respectively.

The performance curves of the 3-storey frames 3ADH and 3ADHsm (Figs. 3a and 3c) obtained by DAP are virtually coincident with those provided by the N2 method with first mode load pattern. In fact, because the fundamental periods of these frames are not large (0,50 for the 3ADH frame and 1,11 s for the 3ADHsm frame) the first mode of vibration basically captures the response and the higher modes provide a negligible contribution. The N2 method applied with the constant load pattern provides a slightly larger base shear demand. Furthermore, both the DAP and the N2 method lead to a base shear demand corresponding to a given top displacement close to that of the IDA. The base shear demand of the 9-storey frames 8CFL and 8CFLsm (Figs. 3b and 3d) obtained by DAP is significantly larger than that given by the N2 method applied with the first mode load pattern. This may be explained by the important contribution given by the higher modes of vibration to the total response of these long period structures, which is taken into account by DAP while it is neglected by the N2 method with the first mode load pattern. If the constant load pattern is considered, DAP method leads to larger base shears compared to the N2 method for the 8CFL frame, and to similar base shears for the 8CFLsm frame. The base shear evaluated by DAP is closer to that given by IDA than that of the N2 method (assumed equal to the maximum between the values obtained by the two load patters) for the 8CFL frame. Instead, both the DAP and N2 method slightly overestimate the actual base shear demand for the 8CFLsm frame.

Figure 4 compares the performance curves in terms of peak ground acceleration a_g and top displacement D_t. For frames 3ADHsm and 8CFLsm (Figs. 4c and 4d), which exhibit a soft storey mechanism, for each value of a_g the DAP and N2 method lead to the same top displacement demand. The relation between a_g and D_t is basically linear and almost identical to that obtained by IDA. The DAP and N2 method provide the same top displacement demand also in the case of the 8CFL frame (Fig. 4b). In fact, its fundamental period is equal to 2.20 s and falls in the displacement sensitivity region. Thus, even in the inelastic range of behaviour, the top displacement increases with the peak ground acceleration according to the same linear relation. Finally, the relation between a_g and D_t is nonlinear for the 3ADH frame (Fig. 4a).

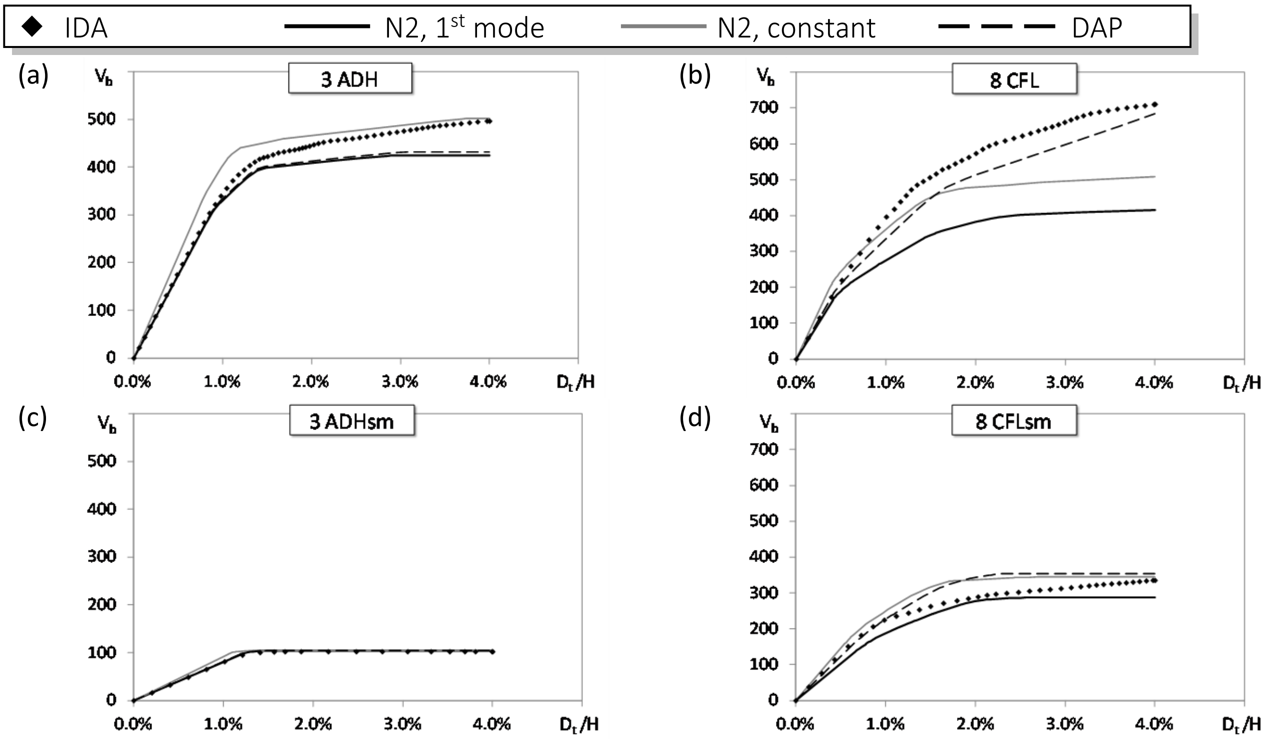


Fig. 3: Capacity curves in terms of base shear and top displacement obtained by DAP, N2 method and IDA: frames with global mechanism (a, b) and soft storey mechanism (c, d).

This result is expected to be common to low period frames that gradually experience inelastic deformation. In these cases because of the period elongation, the slope of the performance curve becomes lower. However, the period elongation is moderate in the case of N2 method and is more significant for DAP. As a consequence, DAP provides larger top displacement demand especially when the frame is well excited in the inelastic range of behaviour. Both the DAP and N2 method lead to a conservative estimation of the top displacement demand given by IDA for the frames 3ADH and 8CFL.

CONCLUSIONS

Modern seismic codes allow nonlinear static analysis to determine the seismic response of existing structures and EC8 suggests the use of N2 method. In this paper a multimodal adaptive variant of N2 method, called DAP, is analysed and the advantages that can be obtained by its application are discussed. This method permits to take into account the effect of higher modes of vibration and to update the modes of vibration at each step of the analysis so to consider the variation of the dynamic properties of the structure. Furthermore, DAP offers an important advantage because it relates directly the required top displacement to the corresponding peak ground acceleration. Thus, it allows a thorough vision of the structural behaviour with reference to several performance objectives, as now required by the most recent seismic codes.

The effectiveness of the proposed method is analysed by comparing the prediction of DAP, in terms of top displacement and base shear demand, to that of N2 method and to the actual response provided by IDA. Both DAP and N2 method predict accurately the base shear corresponding to a given top displacement demand of the two analysed 3-storey frames. The two predictions are very close because the response of these frames is basically not affected by the higher modes of vibration. Instead, DAP provides a base shear demand for the 9-storey frames different from that of the N2 method and generally closer to the actual base shear demand given by IDA. In fact, the response of these frames is significantly affected by the higher modes of vibration, which are taken into account by DAP. For the frames with soft storey mechanism, DAP and N2 method lead to the same prediction of the top displacement required by a given peak ground acceleration and are suitable to predict the actual displacement demand. Finally, both the two considered nonlinear static methods overestimate the top displacement for the frames with global collapse mechanism.

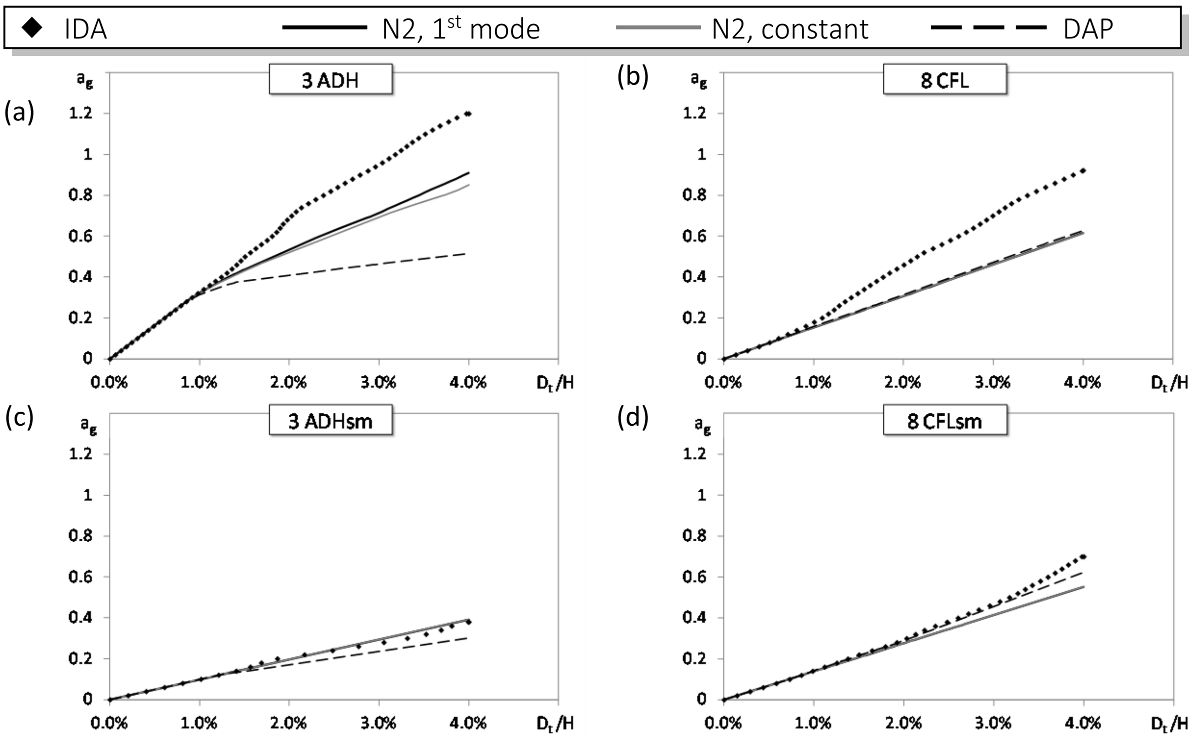


Fig. 4: Capacity curve in terms of pick ground acceleration and top displacement obtained by DAP, N2 method and IDA: frames with global mechanism (a, b) and soft storey mechanism (c, d).

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Title:
Estimation of Seismic Drift and Ductility Demands in Composite Framed Structures: A Design Approach

ESTIMATION OF SEISMIC DRIFT AND DUCTILITY DEMANDS IN COMPOSITE FRAMED STRUCTURES: A DESIGN APPROACH

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ABSTRACT

The seismic inelastic behavior of regular planar composite steel/concrete MRFs consisting of I steel beams and concrete filled steel tube (CFT) columns is investigated. For this purpose, a family of 96 regular plane CFT-MRFs are subject to an ensemble of 100 ordinary (far-field type) ground motions scaled to different intensities in order to accommodate different performance levels and their response to these motions is recorded to form a response databank. On the basis of this databank nonlinear regression analysis is employed in order to derive simple formulae which offer a direct estimation of seismic displacement, drift and ductility demands, and the strength reduction factor q, which describes the seismic strength requirements in order to restrict maximum roof ductility demand to a predefined value. The influence of specific parameters on the maximum structural response, such as the number of stories, the beam-to-column stiffness and strength ratio, the level of inelastic deformation induced by the seismic excitation and the material strengths, is studied in detail. Furthermore, emphasis is given to the ability of the proposed formulae to be adjusted in the framework of seismic design methods which utilize response spectrum analysis. This important design-oriented feature enables both the rapid seismic assessment of existing structures and the direct deformation-controlled seismic design of new ones because the q factor is a function of deformation. A design example reveals the efficiency of the proposed relations in comparison with the procedures adopted in current seismic design codes.

KEYWORDS

Composite frames · Seismic analysis · Drifts · Displacements · Ductilities · Behavior factors

INTRODUCTION

Current earthquake-resistant design codes, in the initial steps of their design process, do not quantify in a direct way the deformation demands of the structures when earthquakes with different magnitudes are applied to them. Instead, it is assumed that in each designed structure according to seismic codes, a known level of stable inelastic deformations can be developed. These deformations are associated with the resistance of the structure and its energy-dissipation capability and related to the extent to which its nonlinear response is capable to take place. During an elastic analysis of a structure, in order to take into account its nonlinear response, a factor in the design stage is used to reduce the forces obtained from a linear analysis. In seismic codes this factor is referred to as strength reduction factor R (UBC, 1997), structural characteristic factor D_s (BSLJ, 2004) or behavior factor q (EC8, 2004).

Nowadays, constant values for the factors R, D_s and q are proposed by seismic codes for different structural types, such as, steel, concrete or steel/concrete composite buildings with moment resisting frames or braced frames e.t.c. The seismic codes consider that through an appropriate reduction factor the structural inelastic deformations will be restricted at the basic limit state, this of human life protection. Moreover, besides this limit state, modern seismic codes (FEMA, 1997) require the satisfaction of two additional limit states: (a) the immediate occupancy in case of minor and more frequent earthquakes, with structural and non-structural elements experiencing no damage; and (b) the collapse prevention in case of the worst probable earthquake, associated with structures prevented to collapse.

Bozorgnia and Bertero (2004) mention that the design method according to EC8 (2004) complies associated with the performance-based design philosophy since different objectives, which usually are target deformations of structural and non-structural elements, should be satisfied for different seismic intensities. In any case, it would be more useful if these deformations were directly used as input parameters with the preliminary design procedure of a structure and not only checked after its seismic analysis. In such a case, pairs of target deformation and intensity of seismic motion are defined for certain limit states and the design base shears are estimated for these limit states. This approach would be realistic if the behavior factor is a function of the drift and ductility demands of the structure for any limit state. This is needed even more for composite frames, where little systematic investigation for their seismic behavior has been carried out, and according to EC8 (2004) the selection of behavior factors for the preliminary design of composite MRFs, is based on those proposed for steel MRFs.

The above leads to the fact that the drift and ductility demands should be determined with sufficient accuracy during the seismic design of building structures and directly associated with the behavior factor through the structural characteristics. The socio-economic losses, the possible building demolition and the human life protection are related to the drift and ductility produced to the buildings by a seismic event. These deformation quantities should be restricted to specific values in order that significant damages on structural and non-structural elements to be avoided.

Thus, the main objective of this paper is to identify and quantify the influence of the various structural characteristics of regular plane CFT-MRFs on their response (drift and ductility demands over their height) to ordinary (i.e., without near fault effects) ground motions. For this purpose, a family of 96 regular plane CFT-MRFs are subject to an ensemble of 100 ordinary ground motions scaled to different intensities in order to accommodate different performance levels and their response to these motions is recorded to form a response databank. Then, nonlinear regression analysis is performed and simple expressions for estimation of seismic displacements, drift and ductility demands are developed. More specifically, in this paper the following objectives are examined: a) the relation between the behavior factor q and the maximum roof displacement ductility, μ; b) a relation that connects the maximum roof displacement $u_{r,max}$ with the maximum inter-storey drift ratio IDR_{max} and c) a relation that correlates μ with the maximum member rotation ductility μ_θ, along the height of the structure. The influence of specific parameters on the maximum structural response, such as the number of stories, the beam-to-column stiffness and strength ratio, the level of inelastic deformation induced by the seismic excitation and the material strengths, is also taken into account. Finally, it is worth noticing that the proposed behavior factor corresponds to the instant of the development of the first plastic hinge in the frame and for this reason, overstrength factors are implicitly considered, while additional static inelastic (pushover) analyses are not required. This essential and design-oriented aspect makes possible both the simple and direct seismic assessment of existing structures and the straightforward (deformation-controlled) seismic design of new ones, taking into account that the factor q is expressed as a function of deformation. The proposed relations are used here in the design procedure of a CFT-MRF and reveal their efficiently and rationality in comparison with the procedures adopted in current seismic design codes.

CFT-MRFs AND GROUND MOTIONS USED IN THIS STUDY

In order to cover a wide range of structural characteristics of CFT-MRFs, a family of 96 plane regular CFT-MRFs are employed for the parametric studies of this work. These frames have storey heights and bay widths equal to 3 m and 5 m, respectively and columns of square concrete filled steel tube (CFT) sections. Moreover, the frames have the following structural characteristics: number of stories, n_s, with values 1, 3, 6, 9, 12, 15, 18 and 20, and three bays, n_b, steel yielding strength ratio $e_s = 235/f_s$ with the yielding stress f_s taking the values of 275 and 355 MPa, concrete strength ratio $e_c = 20/f_c$ with the compressive strength f_c taking the values 20 and 40 MPa (upper and lower limit for dissipative zones according to EC8 (2004)). Additionally, the beam-to-column stiffness ratio ρ (calculated for the storey closest to the mid-height of the frame, Chopra (2007)) and the column-to-beam strength ratio α (taking various values within practical limits) are also considered as follows

$$\rho = \frac{\sum (I/l)_b}{\sum (I/l)_c} \quad , \quad \alpha = \frac{M_{RC,1,av}}{M_{RB,av}} \tag{1}$$

where I and l are the second moment of inertia and length of the steel member (column c or beam b), respectively, $M_{RC,1,av}$ is the average of the plastic moments of resistance of the columns of the first storey and $M_{RB,av}$ is the average of the plastic moments of resistance of the beams of all the stories of the frame.

Every frame is designed for vertical static loads according to EC3 (2005) and 4 (2004) and for seismic loads according to EC8 (2008) using design ground acceleration $\alpha_g = 0.30$g, soil type B (soil factor $S = 1.2$) and Spectrum Type 1 with behavior factor $q = 4$. In addition to the satisfaction of the seismic strength demands in members, other seismic design checks include compliance with stability and drift criteria as well as capacity design considerations. Then, an ensemble of 100 ordinary (far-field type) ground motions of soil type B with their geometric average spectrum be as near as possible to the EC8 (2004) elastic spectrum for ground acceleration 0.30 g are selected without any scaling for performing the nonlinear time history analyses of this study. The acceleration response spectra of the selected 100 motions are shown in Figure 1 against to the EC8 (2004) elastic spectrum. A full list of all these ground motions and designed frames with their characteristics can be found in Skalomenos (2014).

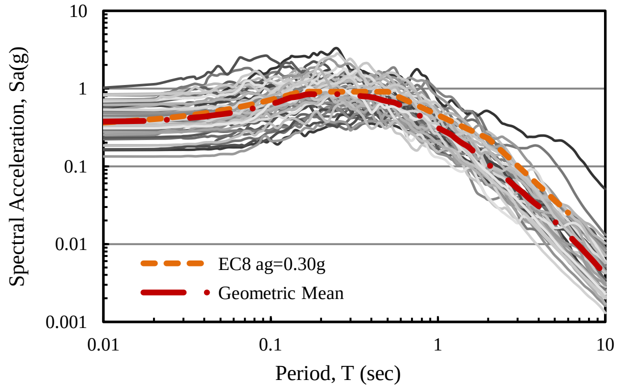

Figure 1 Acceleration response spectra of the 100 earthquakes and comparison with EC 8 (2004) elastic spectrum.

MODELLING FOR NON-LINEAR ANALYSES

The 96 CFT-MRFs mentioned in the previous section, are subjected to a set of 100 accelerograms and their response to those motions is determined through inelastic dynamic analyses with the aid of the computer program RUAUMOKO (Carr, 2006). Diaphragm action is assumed at every floor due to the presence of the slab, the effect of large deformations is taken into account and Rayleigh damping (Chopra, 2007) corresponding to 3% of the critical damping is considered. The deteriorating inelastic behavior of all the frame members is modeled by means of hysteretic point plastic hinges. The effect of panel zones is taken into account and the connections are assumed to be rigid. The analytical models of frame components utilized here are presented in detail in Skalomenos et al. (2014a). The considered here analytical models account for nonlinearities due to yielding and local buckling of the steel beams and steel tubes of the CFT columns, cracking and crushing of concrete in the CFT columns and steel yielding and concrete cracking and crushing in the panel zones. Finally, the modelling/analysis take into account the M - N interaction diagram, such as the one described in Skalomenos (2014) according to EC4 (2004).

PARAMETRIC STUDIES AND RESPONSE DATABANK

The response databank of the 96 CFT-MRFs is generated by thousands of dynamic nonlinear time histories. These analyses are based on incremental dynamic analysis (IDA). IDA involves performing nonlinear dynamic analyses in the structural model under a suite of ground motion records, each scaled to several intensity levels designed to force the structure all the way from elastic behavior to final global dynamic instability and collapse (Vamvatsikos and Cornell 2002). For every structure and accelerogram pair, the scale factors (*SF*) of the accelerogram which correspond to five performance levels are identified and recorded. Those six performance levels (SEAOC 1999; FEMA 1997) are: a) occurrence of the first plastic hinge, b) IDR_{max} = 0.5%, c) IDR_{max} = 1.8%, d) IDR_{max} = 2.5%, e) IDR_{max} = 3.2% and f) IDR_{max} = 4.0%. The results of the nonlinear dynamic analyses (100 accelerograms × 96 frames × 6 elastic and inelastic performance levels × 10 iterations = 576000 analyses) are post-processed in order to create a databank with the response quantities of interest, i.e., IDR_{max}, $u_{r,max}$, μ_θ, the roof displacement at first yielding $u_{r,y}$, $\mu = u_{r,max}/u_{r,y}$ and q. The behavior factor q is calculated as the ratio of the *SF* of the accelerogram which leads the structure to a specific performance level over the *SF* of the accelerogram which corresponds to first yielding, based on nonlinear analysis results (Elnashai and Broderick 1996; Mazzolani and Piluso, 1996; Karavasilis et al. 2008). For instance, for the fifth performance level with the *SF* values that correspond to the occurrence of the first plastic hinge denoted as SF_y and those to IDR = 3.2% denoted as $SF_{3.2}$ being known, the behavior factor can be directly calculated as $q = SF_{3.2}/SF_y$.

ESTIMATION OF SEISMIC DISPLACEMENTS, DRIFT AND DUCTILITY DEMANDS

In this section, formulae to estimate seismic displacements, drift and ductility demands in regular plane CFT-MRFs are proposed. With R being any response quantity of interest, the calculated sample median, mean and the standard deviation of the ratio of the approximate to the ‘exact’ value of R, i.e., R_{app} / R_{exact}, are used in order to express the central tendency and the dispersion of the error introduced by the proposed relations. The ‘exact’ value corresponds to the results of dynamic inelastic analyses and does not mean that any type of uncertainties or randomness has been eliminated.

Estimation of the maximum roof displacement ductility by using the target IDR

The calculation of the roof displacement ductility by using the target *IDR* of a predefined performance level, is achieved by using the following equation,

$$\mu_{r,IDR} = \frac{u_{r,max(IDR)}}{u_{r,y}} \tag{2}$$

where $u_{r,max(IDR)}$ is the maximum roof displacement for the predefined *IDR*. The $u_{r,y}$ can be estimated by forlmulae proposed in Skalomenos (2014b). Medina and Krawinkler (2005) proposed a relation which correlates the $u_{r,max(IDR)}/H$ with the IDR_{max} and depends on the fundamental period of vibration of the frame. Here, by analyzing the response databank the ratio $\beta = u_{r,max(IDR)}$ / (H·IDR_{max}) was found to depend on the number of stories n_s, the parameters ρ, α and the grade of steel f_s, but no significant dependence on the concrete strength f_c has been observed. Thus, by using a nonlinear regression analysis (MATLAB, 2009) the following approximation for the ratio β is produced

$$\beta = 1 - 0.547 \cdot \left(n_s^{0.259} - 1\right) \cdot \rho^{-0.154} \cdot \alpha^{-0.334} \cdot e_s^{-0.130} \tag{3}$$

Equation (6) is simple, the fundamental and rational principle $\beta = 1$ for $n_s = 1$ is satisfied and by using this equation one can estimate the $u_{r,max}$ given the IDR_{max} and vice versa. With the inter-storey drift ratio known, Eq. (3) offers a ratio $u_{r,max,app}/u_{r,max,exact}$ with a mean value equal to 1.0, a median value equal to 1.0 and dispersion equal to 0.12 Finally, the correlation factor R^2 between the median of ‘exact’ values $u_{r,max,median,exact}$ of the databank with those resulting from the proposed Eq. (3) is equal to 0.994.

Estimation of the maximum roof displacement ductility by using the target rotation ductility

In this section, an effort to correlate the maximum rotation ductility μ_θ, which is evaluated at the end of beams and columns, with the maximum roof displacement ductility μ, is made. The relation between μ and μ_θ was found to depend on the number of stories, the parameters ρ, α and the grade of steel f_s of the frame. Thus, the nonlinear regression analysis provided the following relation

$$\mu_{r,\theta} = 1 + 1.078 \cdot \left(\mu_\theta^{0.936} - 1\right) \cdot n_s^{-0.044} \cdot \rho^{0.144} \cdot \alpha^{0.375} \cdot e_s^{0.211} \tag{4}$$

Equation (4) appears to be simple and satisfies the fundamental and rational principle $\mu = 1.0$ for $\mu_\theta = 1.0$. In case the maximum rotation ductility μ_θ is known, Eq. (4) offers a ratio $\mu_{r\theta,app} / \mu_{r\theta,exact}$ with a mean value equal to 1.0, median value equal to 1.0 and dispersion equal to 0.15. Finally, the correlation factor between the median of 'exact' values $\mu_{r\theta,median,exact}$ of the databank with those resulting by the proposed Eq. (4) is equal to 0.991.

Thus, from the above equations, the roof displacement ductility demand μ_r is defined as the minimum value of $\mu_{r,IDR}$ and $\mu_{r,\theta}$,

$$\mu_r = min\left(\mu_{r,IDR}, \mu_{r,\theta}\right) \tag{6}$$

Estimation of the behavior factor q

The EC8 (2004) adopts the equal displacement rule for estimating the maximum inelastic roof displacement, i.e. $u_{r,max} = q \cdot u_{r,y}$ with $q = \mu_r$. By using the response databank here, this rule offers a ratio $\mu_{r,app} / \mu_{r,exact}$ with mean value equal to 1.17, median value equal to 1.10 and dispersion equal to 0.3. During sensitivity analysis of the response databank, it was identified that the frame structural or the ground motion characteristics do not affect the relationship between q and μ_r. Thus, the nonlinear regression analysis gives the following formula

$$q = 1 + 1.90 \cdot \left(\mu_r^{0.76} - 1\right) \tag{7}$$

The aforementioned relation is relatively simple and satisfies the fundamental and rational principle $q = 1$ for $\mu_r = 1$. In addition, one can determine directly the behavior factor q for a predefined roof ductility value, or alternatively, for known yield roof displacement, $u_{r,y}$, it leads to the evaluation of the maximum inelastic roof displacement $u_{r,max}$.

Examining Eq. (7), it should be mentioned that the ratio $\mu_{r,app} / \mu_{r,exact}$ is computed with a mean value equal to 1.04 and median value equal to 1.03, while the dispersion is found equal to 0.22. Figure 2a depicts the 'exact' results (from dynamic inelastic analysis) of the databank, the proposed empirical expression of Eq. (7) and the hypothesis of EC8 (2004), i.e., $q = \mu_r$. Furthermore, Figure 2a depicts the approximation of the databank with the proposed equation of Karavasilis et al (2008) which was developed for pure steel plane MRFs, in order to compare their behavior with those of CFT-MRFs.

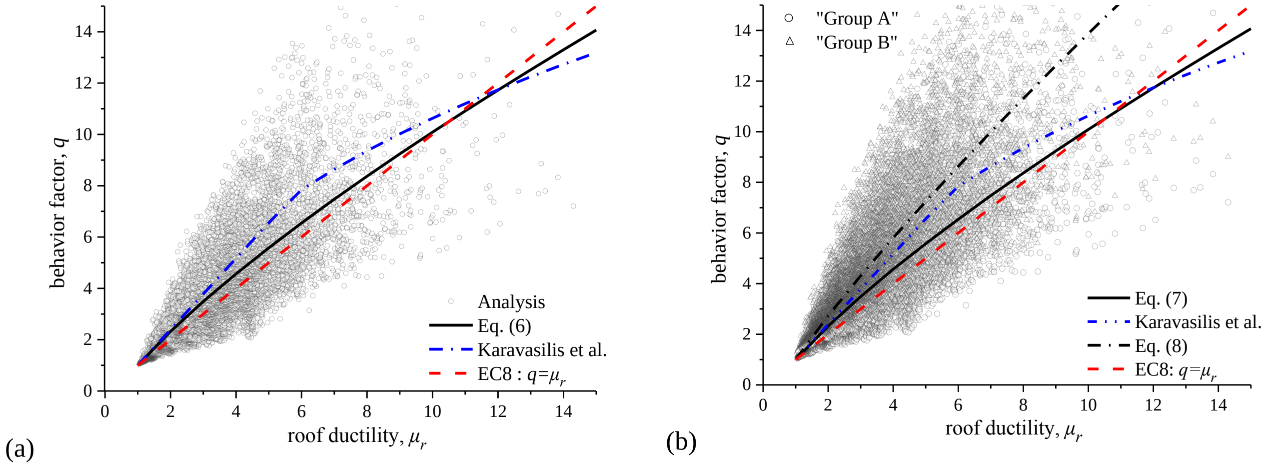

Figure 2 (a) Graphical approximation of the response databank with the proposed Eq. (7), the assumption of EC8 (2004) $q = \mu_r$ and the proposed equation of Karavasilis et al. (2008); (b) graphical approximation of the response databank with the proposed Eq. (7) and (8), the assumption of EC8 (2004) and the proposed equation of Karavasilis et al. (2008).

From the above figure, it is obvious that the equal-displacement rule ($q = \mu_r$) clearly over-estimates the 'exact' maximum roof ductility demands in both CFT-MRFs and steel MRFs. In addition, the main observation from Figure 2a has to do with the difference between the behavior of CFT-MRFs and pure steel MRFs. For the same behavior factor q, the CFT-MRFs seem to require higher ductility demands than the steel MRFs. However, this is not true and should be noted that these two different types of structures have been analyzed on the basis of models with different levels of sophistication. More specifically, the proposed equation of Karavasilis et al. (2008), has been derived by disregarding panel zones and deterioration on beams and columns. In contrast, the present analyses were based on detailed modelling of CFT-MRFs with all the important inelastic effects of their individual components taken into account, as discussed above. Thus, it was therefore considered appropriate that the analyses for all CFT-MRFs based on the assumptions of Karavasilis et al. (2008a), should be repeated in order to have a realistic comparison. After analyzing the new response databank following approximation is produced

$$q = 1 + 3.022 \cdot \left(\mu_r^{0.687} - 1\right) \tag{8}$$

Figure 2b portrays schematically the approximation of the response databank with the proposed Eq. (7), Eq. (8) which is based on the assumptions of Karavasilis et al. (2008), the proposed equation of Karavasilis et al (2008) and the assumption of EC8 (2004) $q = \mu_r$. Furthermore, in this figure the term "Group A" refers to the databank corresponding to the response of CFT-MRFs with sophisticated modelling (considering deterioration effects on steel beams and columns and panel zone deformation), while the term "Group B" corresponds to the CFT-MRFs modelled using the same assumptions as Karavasilis et al. (2008) (ignoring deterioration effects on steel beams and columns and panel zone deformation).

The results of the additional analyses for the cases of CFT-MRFs modelled after the assumptions of Karavasilis et al. (2008), confirm that the great difference for the ductility demands between the proposed Eqs (7) and (8) has to do with the modelling level. It is obvious now, that Eq. (8) can be compared with that proposed by Karavasilis et al. (2008) on an equal basis because both of them have been developed with the same assumptions in modelling. Thus, for the same factor q, the CFT-MRFs seem to have better seismic behavior than the pure steel MRFs since they are associated with smaller ductility demands, or for the same ductility demands, the CFT-MRFs appear to be more efficient than the pure steel MRFs since they are associated with higher behavior factors. It should be noted that EC8 (2004) sets identical behavior factor for steel and for composite MRFs. However, the above finding clearly indicates the necessity of providing different behavior factors for composite and pure steel structures in the seismic code. This finding is also consistent with that of Elnashai and Broderick (1996) who recommend a 19-40% increase in the behavior factors provided by EC8 (1994) for steel structures in order to design composite ones. Here, Eq. (8) suggests approximately a 18% increase in the behavior factors, which means a further reduction on the seismic design loads for CFT-MRFs by 15%, in comparison with those factors proposed by Karavasilis et al. (2008) for pure steel MRFs. This comparison is made for reasonable values of ductility demands, e.g., $\mu_r < 6$ (Figs 15, 16).

On the basis of Figure 2b, the importance of an appropriate selection of the modelling level is apparent, for it can lead to a different seismic behavior and different conclusions for the same seismic intensity and structure. It should be noted at this point that the additional analyses and work by Skalomenos et al. (2014b) revealed the necessity to take into account more sophisticated models to simulate the seismic behavior of CFT-MRFs. Moreover, this sophisticated modelling explains the difference between the curves obtained by using the proposed Eqs (7) and (8). The determination of the behavior factor q through Eq. (7) is proposed here as more conservative than that of Eq. (8), since the strength and stiffness deterioration leads to a smaller scale factor of seismic intensity and therefore to a smaller factor q, as shown in Figure 2b.

A SIMPLE DESIGN EXAMPLE: USE OF THE PROPOSED RELATION

The following example shows that the relations presented in this paper may be an effective tool for dimensioning a structure on the basis of a simple and fast response spectrum analysis. Then, any other rigorous inelastic procedure may be used to validate or to change the product of the design process.

A three-bay and five-storey CFT-MRF is designed by steel yielding stress f_s equal to 275MPa and concrete compressive strength f_c equal to 20MPa. The bay width is assumed equal to 6 m and the storey height equal to 3 m. The gravity load on beams is equal to 27.5 kN/m. The design ground motion is defined by the elastic acceleration response spectrum of the EC8 (2004) seismic code with a PGA equal to 0.35 g and a soil class B. The design of the frame is made according to the EC3 (2005), EC4 (2004) and EC8 (2004) structural codes with the aid of the commercial software package SAP2000 (2013). A q factor equal to 3.0 is selected and the design procedure yields the following sections for the five stories of the frame, going for the first to the fifth storey: (a) CFT420x20–IPE400, (b) CFT420x20–IPE450, (c) CFT400x16–IPE450, (d) CFT350x16–IPE400 and (e) CFT320x16–IPE360. The symbolism of, e.g., CFT420×20–IPE400, means that the floor has CFT columns with square steel tubes of width b = 420 mm and thickness t = 20 mm and IPE400 beams. The maximum roof displacement and inter-storey drift ratio under the reduced (divided by q) spectrum are equal to 0.12 m and 0.006, respectively. Thus, according to EC8 (2004) the maximum inelastic roof displacement equals $3.0 \cdot 0.12 = 0.36$ m and the maximum inelastic inter-storey drift $3.0 \cdot 0.006 = 0.018$.

On the basis of the proposed relations, Eq (1) determines the values of parameters ρ and α of the designed frame equal to 0.197 and 3.63, respectively, while according to Skalomenos et al. (2014b) the roof displacement at first yield $u_{r,y}$ is equal to 0.11 m. Given the value 3.0 for the behavior factor q, Eq. (7) estimates the maximum roof displacement ductility μ_r equal to 2.50 and therefore, according to Eq. (2) the target roof displacement $u_{r,max(IDR)}$ is found to be equal to $0.11 \cdot 2.50 = 0.275$ m. Finally, by employing Eq. (3) the ratio β is calculated equal to 0.76 and thus, IDR_{max} is predicted equal to 0.024.

In order to compare the results of the EC8 (2004) design procedure with those of the proposed one, nonlinear time history analyses of the designed frame are performed. For this purpose, ten semi-artificial accelerograms compatible with the EC8 spectrum are generated via a deterministic approach (Karabalis et al., 1993). The analysis results under these motions yield the following mean values: $u_{r,max} = 0.25$, $IDR_{max} = 0.025$, $\mu_r = 2.40$ and $q = 2.95$ and therefore, reveal that the proposed procedure seems to be more rational and efficient than the procedure of EC8 (2004), for performance-based seismic design of CFT-MRFs.

SUMMARY AND CONCLUSIONS ON THE PROPOSED EMPIRICAL EXPRESSIONS

An efficient procedure in terms of simple formulae for estimating global and local seismic drift and ductility demands in regular plane CFT-MRF buildings subjected to ordinary (i.e., without near-fault effects) earthquake ground motions has been presented. These formulae were developed on the basis of the results of thousand time history analyses. Moreover, additional analyses for CFT-MRFs with simpler modelling sophistication in agreement with those assumptions adopted by researchers who investigated the seismic inelastic behavior of pure steel frames were executed in order for the comparison between the seismic behavior of CFT-MRFs and that of pure steel frames previously studied by other authors to be possible. All the proposed equations are simple, satisfy the physical constraints, express the central tendency with small dispersion and reflect the influence of the structural parameters (a, ρ, n_s, e_s e.t.c.). On the basis of the developments presented in this study, the following conclusions can be drawn:

(1) The q factor expressions proposed here correspond to the point of first yielding in the frame and thus, pushover analysis and estimation of the overstrength factor are not needed. This important feature enables both the rapid seismic assessment of existing structures and the direct deformation-controlled seismic design of new ones.

(2) The comparison between the Eqs (7) and (8) for q revealed the strong influence of the appropriate selection of modelling level which can lead to different seismic behavior of frames and to different conclusions. For example, strength and stiffness deterioration lead to a smaller scale factor for seismic intensity and therefore to a smaller factor q.

(3) The comparison of Eq. (8) for q against the equations proposed for pure steel MRFs revealed that the CFT-MRFs seem to have better seismic behavior than the pure steel MRFs since they are associated with smaller ductility demands and higher behavior factors. This finding clearly indicates that behavior factors of CFT-MRFs and steel MRFs should not to be the same in the seismic codes.

(4) Eq. (8) suggests approximately an 18% increase in the behavior factors of CFT-MRFs in comparison with those proposed for steel MRFs. This means a further reduction of the seismic design loads for CFT-MRFs by 15%, allowing some advantages to be gained from the benefits of this type of structures.

(5) Reliable estimates of the seismic displacements, drift and ductility demands can be provided by the proposed formulae. Compared with the procedure adopted by current seismic design codes, it is found to be more rational and efficient for performance-based seismic design and/or assessment of regular plane CFT-MRFs buildings.

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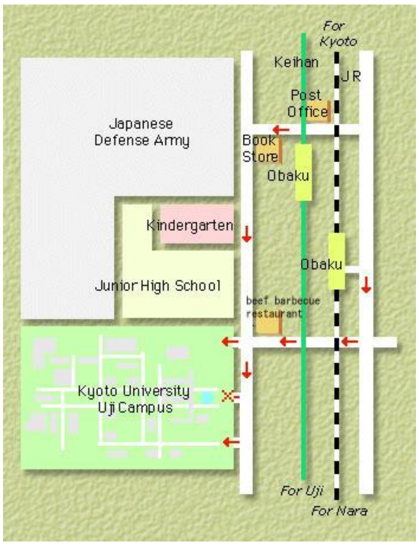

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