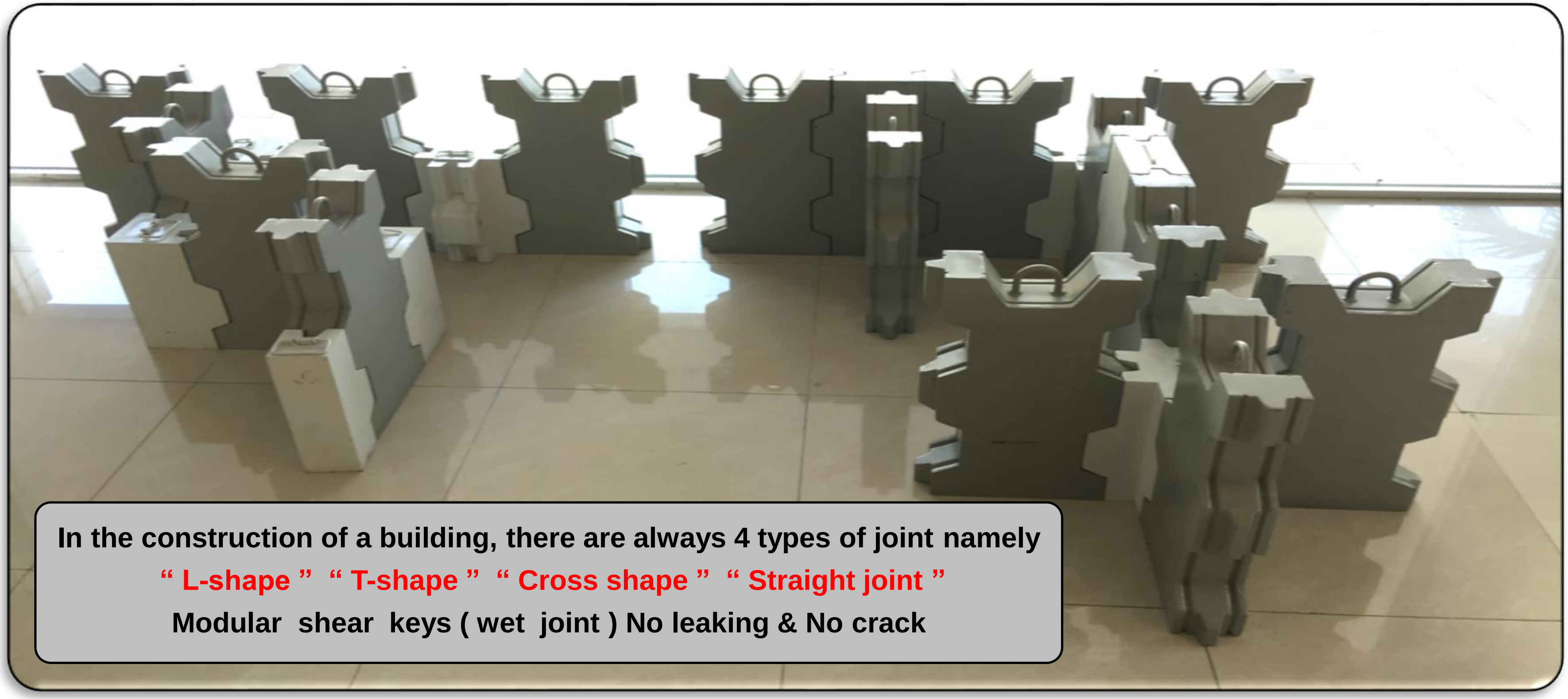
HC Precast System

(100 % Malaysia Technology With 6 IPs')

Economical . Eco Friendly . Quality

- **100% Malaysian Technology with 6 IP's (Intellectual Property)**
- **Internationally Published by Foreign Journals and Magazines**
- **Customized & Flexibility to suit all Architectural Demands**
- **Designed for Tropical Climate Country by using " Wet Joint " system Earthquake**



In the construction of a building, there are always 4 types of joint namely

" L-shape " " T-shape " " Cross shape " " Straight joint "

Modular shear keys (wet joint) No leaking & No crack

Economical . Eco Friendly . Quality

The system is a proprietary technology that has been established in accordance to British Standards (BSI) and is also a patented technology.

The main design of the connection system has also been subjected to detail checking by an Independent Checker.

Hence, the specifications are not to be altered without proper engineering study to ensure the safety and integrity of the precast system.

- **United States Patent : US 6,829,870 B2**
- **Malaysia Patent : MY - 124213 - A**
- **Malaysia Patent : MY - 139712 - A**
- **Malaysia Patent : MY - 157696 - A**
- **Malaysia Patent : MY - 162115 - A**
- **Republic Of Indonesia Patent : IDP 000047693**

The propriety ship of HCPS system has been internationally published by foreign journals and magazines.

- **Case Studies in Structure Engineering (ELSVIER) : UK**
- **Earthquakes and Structures : USA**
- **Concrete Plant International (CPI) : German (8 difference language)**
- **15^{th} World Conference on Earthquake : Lisbon & Portugal**
- **Shake Table Test of 1:3 Scaled HCPS Precast Concrete Wall System : Journal by Nanyang Technical University**

United States Patent
US 6,829,870 B2

The United States of America

The Director of the United States Patent and Trademark Office

Has received an application for a patent for a new and useful invention. The title and description of the invention are enclosed. The requirements of law have been complied with, and it has been determined that a patent on the invention shall be granted under the law.

Therefore, this

United States Patent

Grants to the person(s) having title to this patent the right to exclude others from making, using, offering for sale, or selling the invention throughout the United States of America or importing the invention into the United States of America for the term set forth below, subject to the payment of maintenance fees as provided by law.

If this application was filed prior to June 8, 1995, the term of this patent is the longer of seventeen years from the date of grant of this patent or twenty years from the earliest effective U.S. filing date of the application, subject to any statutory extension.

If this application was filed on or after June 8, 1995, the term of this patent is twenty years from the U.S. filing date, subject to any statutory extension. If the application contains a specific reference to an earlier filed application or applications under 35 U.S.C. 120, 121 or 365(c), the term of the patent is twenty years from the date on which the earliest application was filed, subject to any statutory extensions.

Jon W. Dudas

Director of the United States Patent and Trademark Office

United States Patent
US 6,829,870 B2

US006829870B2

(12) **United States Patent**
Hur

(10) **Patent No.: US 6,829,870 B2**
(45) **Date of Patent: Dec. 14, 2004**

(54) **BUILDING METHODS**

(75) Inventor: **Teow Beng Hur**, Selangor Darul Ehasn (MY)

(73) Assignee: **HC Precast System SDN. BHD**, Selangor Darul Ehasn (MY)

(*) Notice: Subject to any disclaimer, the term of this patent is extended or adjusted under 35 U.S.C. 154(b) by 16 days.

(21) Appl. No.: **10/285,548**

(22) Filed: **Nov. 1, 2002**

(65) **Prior Publication Data**

US 2004/0016199 A1 Jan. 29, 2004

(51) **Int. Cl.**[7] **E04C 2/38**
(52) **U.S. Cl.** **52/656.1**; 52/563; 52/631; 249/47; 249/191
(58) **Field of Search** 249/19, 22, 26, 249/27, 47, 191; 52/426, 562, 563, 275, 656.1, 631

(56) **References Cited**

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* cited by examiner

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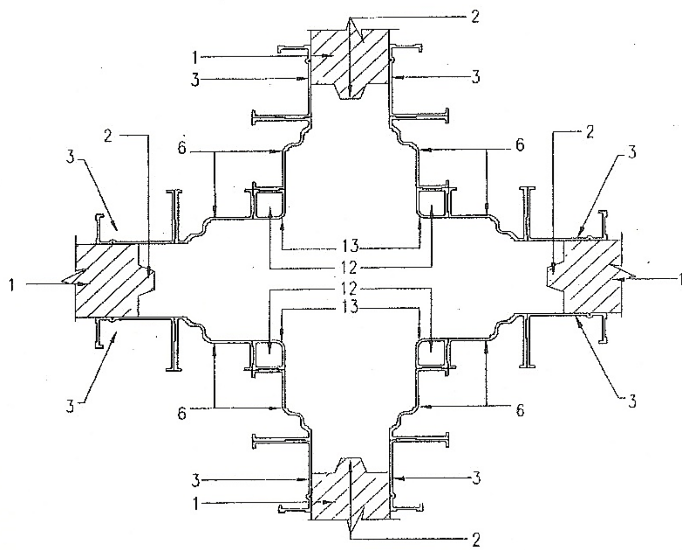
(57) **ABSTRACT**

A building is erected using pre-cast wall panels, preferably load-bearing wall panels having a shear key on each vertical edge and starter bars on each horizontal edge, by first erecting the wall panels, and then casting a concrete column around the vertical edges of adjacent or intersecting wall panels using movable formwork made up from a set of standard modules that can be assembled to form different configurations and sizes of column for different panel arrangements. The moulding surfaces of the modules may be shaped to provide decorative features to the columns and/or the column/wall intersections.

3 Claims, 5 Drawing Sheets



Malaysia
MY - 124213 - A

MALAYSIA

CERTIFICATE OF GRANT OF A PATENT

In accordance with Section 31(2) of the Patents Act 1983 a patent for an invention having grant number MY - 124213 - A has been granted to HC PRECAST SYSTEM SDN BHD in respect of an invention having the following particulars :

TITLE	:	IMPROVEMENTS IN BUILDING METHODS.
FILING DATE	:	25 JULY 2002
PRIORITY DATE	:	NONE
NAME OF INVENTOR	:	TEOW BENG HUR.
PATENT OWNER	:	HC PRECAST SYSTEM SDN BHD : NO. 1, JALAN SINGA 20/E , SEKSYEN 20, 40000 SHAH ALAM, SELANGOR DARUL EHSAN, MALAYSIA.
DATE OF GRANT	:	30 JUNE 2006

Dated this 30 day of JUNE 2006

(MOHD. AMRAN BIN ABAS)
for Registrar of Patents
MALAYSIA

Malaysia
MY - 139712 - A

MALAYSIA

CERTIFICATE OF GRANT OF A PATENT

In accordance with Section 31 (2) of the Patents Act 1983 a patent for an invention having grant number MY - 139712 - A has been granted to HC PRECAST SYSTEM SDN. BHD. in respect of an invention having the following particulars :

TITLE	:	PANEL FORMWORK SYSTEM
FILING DATE	:	27 MAY 2003
PRIORITY DATE	:	NONE
NAME OF INVENTOR	:	TEOW BENG HUR
PATENT OWNER	:	HC PRECAST SYSTEM SDN. BHD. NO. 1, (GRD. FLOOR) JALAN SINGA 20/E SEKSYEN 20 40000 SHAH ALAM SELANGOR DARUL EHSAN MALAYSIA
DATE OF GRANT	:	30 OCTOBER 2009

Dated this 30 day of OCTOBER 2009

Shamsiah

(SHAMSIAH BINTI KAMARUDDIN)
for Registrar of Patents
MALAYSIA

Malaysia
MY - 157696 - A

MALAYSIA

CERTIFICATE OF GRANT OF A PATENT

In accordance with Section 31 (2) of the Patents Act 1983 a patent for an invention having grant number MY-157696-A has been granted to HC PRECAST SYSTEM SDN. BHD., in respect of an invention having the following particulars :

TITLE	:	EARTHQUAKE PROOF WALL PANELS
FILING DATE	:	22 JULY 2010
PRIORITY DATE	:	NONE
NAME OF INVENTOR	:	TEOW BENG HUR
PATENT OWNER	:	HC PRECAST SYSTEM SDN. BHD. NO. 23B, JALAN SERI SARAWAK 20B/KS 2, TAMAN SRI ANDALAS 41200 KLANG SELANGOR DARUL EHSAN MALAYSIA
DATE OF GRANT	:	15 JULY 2016
DURATION OF PATENT	:	22 JULY 2010 UNTIL 22 JULY 2030
END OF PROTECTION	:	14 JULY 2017 (SUBSEQUENT ANNUAL FEE SHALL FOLLOW AS STATED IN THE SCHEDULE OF FEES AT THE BACK OF THIS PAGE)

Dated this 15 day of JULY 2016

(DATO' SHAMSIAH BINTI KAMARUDDIN)
Registrar of Patents
MALAYSIA

Malaysia
MY - 162115 - A

MALAYSIA

CERTIFICATE OF GRANT OF A PATENT

In accordance with Section 31 (2) of the Patents Act 1983 a patent for an invention having grant number MY-162115-A has been granted to HC PRECAST SYSTEM SDN. BHD., in respect of an invention having the following particulars :

TITLE	:	BEAM MOULD SUPPORT ASSEMBLY
FILING DATE	:	12 FEBRUARY 2009
PRIORITY DATE	:	NONE
NAME OF INVENTOR	:	TEOW BENG HUR
PATENT OWNER	:	HC PRECAST SYSTEM SDN. BHD. NO. 23B, JALAN SERI SARAWAK 20B/KS 2 TAMAN SRI ANDALAS 41200 KLANG SELANGOR DARUL EHSAN MALAYSIA
DATE OF GRANT	:	31 MAY 2017
DURATION OF PATENT	:	12 FEBRUARY 2009 UNTIL 12 FEBRUARY 2029
END OF PROTECTION	:	30 MAY 2018 (SUBSEQUENT ANNUAL FEE SHALL FOLLOW AS STATED IN THE SCHEDULE OF FEES AT THE BACK OF THIS PAGE)

Dated this 31 day of MAY 2017

(DATO' SHAMSIAH BINTI KAMARUDDIN)
Registrar of Patents
MALAYSIA

Republik Indonesia
IDP000047693

REPUBLIK INDONESIA
KEMENTERIAN HUKUM DAN HAK ASASI MANUSIA

SERTIFIKAT PATEN

Menteri Hukum dan Hak Asasi Manusia atas nama Negara Republik Indonesia berdasarkan Undang-Undang Nomor 13 Tahun 2016 tentang Paten, memberikan Paten kepada:

Nama dan Alamat Pemegang Paten	:	HC PRECAST SYSTEM Sdn. Bhd No. 23B, Jalan Seri Sarawak 20B/KS2, Taman Sri Andalas, 41200 Klang, Selangor MALAYSIA
Untuk Invensi dengan Judul	:	PANEL DINDING YANG TAHAN TERHADAP GEMPA BUMI
Inventor	:	Teow Beng Hur
Tanggal Penerimaan	:	20 Desember 2010
Nomor Paten	:	IDP000047693
Tanggal Pemberian	:	07 September 2017

Perlindungan Paten untuk invensi tersebut diberikan untuk selama 20 tahun terhitung sejak Tanggal Penerimaan (Pasal 22 Undang-Undang Nomor 13 Tahun 2016 tentang Paten).

Sertifikat Paten ini dilampiri dengan deskripsi, klaim, abstrak dan gambar (jika ada) dari invensi yang tidak terpisahkan dari sertifikat ini.

00-2017-287880

a.n. MENTERI HUKUM DAN HAK ASASI MANUSIA
REPUBLIK INDONESIA
DIREKTUR JENDERAL KEKAYAAN INTELEKTUAL
u.b.
Direktur Paten, Desain Tata Letak
Sirkuit Terpadu dan Rahasia Dagang,

Dra. Dede Mia Yusanti, M.L.S
NIP. 196407051992032001

Republik Indonesia
IDP000047693

MINISTRY OF LAW AND HUMAN RIGHTS
OF THE REPUBLIC OF INDONESIA

PATENT CERTIFICATE

Minister of Law and Human rights, in the name of the Republic of Indonesia, in accordance to the Law Number 13 of 2016 concerning Patents, has granted a Patent to :

Name and Address of The Patent Holder	:	HC PRECAST SYSTEM Sdn. Bhd. No. 23B, Jalan Seri Sarawak 20B/KS2, Taman Sri Andalas, 41200 Klang, Selangor, MALAYSIA
for the Invention :		
Entitled	:	EARTHQUAKE PROOF WALL PANELS
The Inventors	:	Teow Beng Hur
Date of receipt of the application	:	December 20, 2010
Patent Number	:	IDP000047693
Date of Grant	:	September 7, 2017

Patent Protection for the Invention shall be granted for a period of 20 years as of the date of receipt of this patent application (Article 22 of Law No. 13 of 2016 concerning patents).

This Patent Certificate is enclosed with the description, claims, abstract and drawing(s) (if any) that are inseparable parts of this Patent Certificate.

BARCODE

00-2017-287880

For : THE MINISTER OF LAW AND HUMAN RIGHTS OF
THE REPUBLIC OF INDONESIA
DIRECTOR GENERAL OF INTELLECTUAL PROPERTY RIGHTS
Patent Director, Layout Design of
Integrated Circuit and Trade Secret

(Signed)

Dra Dede Mia Yusanti, M.L.S
NIP. 196407051992032001

Republik Indonesia
IDP000047693

(12) PATEN INDONESIA

(19) DIREKTORAT JENDERAL KEKAYAAN INTELEKTUAL

(11) IDP000047633 B

(45) 07 September 2017

(51) Klasifikasi IPC[8] : E 04H 9/02

(21) No. Permohonan Paten : P00201000903

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0) Data Prioritas :

(31) Nomor	(32) Tanggal	(33) Negara
PI2010003480	22 Juli 2010	MY

Tanggal Pengumuman: 26 Januari 2012

Dokumen Pembanding:
US 5678372 A
JS 2739364 A

(71) Nama dan Alamat yang Mengajukan Permohonan Paten :
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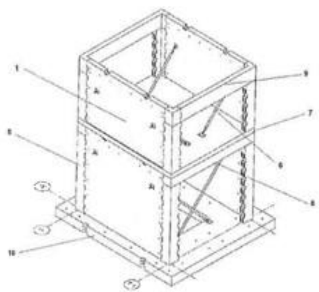
Pemeriksa Paten : Aditia Meiriza Ashibi, ST.

Jumlah Klaim : 5

vensi : PANEL DINDING YANG TAHAN TERHADAP GEMPA BUMI

vensi ini berhubungan dengan panel dinding pracetak beton bertulang (1) yang membentuk elemen struktural dari struktur dan memungkinkan rancangan dan membangun suatu bangunan yang menggunakan pendekatan modular. Pada panel (1) 'umlah ceruk (3) dan sejumlah tonjolan kunci berceruk (4) yang berselang-seling secara bergantian dengan satu sama lain di 'ua tepi sisi panel dinding yang berhadapan dan komponen bertulang (2) yang menonjol dari sejumlah ceruk (3). Pada panel, panel dinding yang berhadapan dengan sejumlah ceruk (3) dan sejumlah tonjolan kunci berceruk (4) dicerminkan dengan satu hingga sejumlah ceruk (3) dan sejumlah tonjolan kunci berceruk (4) pada salah satu dari kedua tepi sisi panel dinding yang lsejajarkan secara horizontal dengan sejumlah ceruk (3) dan sejumlah tonjolan kunci berceruk (4) pada salah satu dari kedua dinding yang berhadapan lainnya. Selanjutnya, komponen bertulang (2) membentang dengan panjang yang ditentukan lebih kedalaman sejumlah ceruk (3).



UK

ELSEVIER

Case Studies in Structural Engineering

Certificate of publication for the article titled:

"Case study of load-bearing precast wall system subject to low seismic intensity by linear and nonlinear analyses "

Authored by:

Patrick Liq Yee Tiong, Sing Ping Chiew and Beng Hur Teow

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Case study of load-bearing precast wall system subject to low seismic intensity by linear and nonlinear analyses

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Pushover
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ABSTRACT

This paper investigates the behaviour of load-bearing precast wall system (namely the HC Precast System – HCPS) subject to seismicity in Malaysia. Recent tremors felt across the country heeded the call for the need of seismic design guidelines to be implemented. For this study, the design ground acceleration for Malaysia has not been finalized. Throughout the years, several schools of thought that occurred among different researchers pertaining to the value of design ground acceleration, ranging from 0.05 g to 0.1 g. The implications of the selected values can be great especially in designing new buildings or retrofitting existing ones. Thus, linear analysis using Modal Response Spectrum Analysis (MRSA) and nonlinear pushover analysis of representative HCPS were performed for this study. The finite element (FE) model focused particularly on the nonlinear behaviour of the interface between a precast wall and cast in-situ column. Prior to the modal and pushover analyses, the FE model was validated against quasi-static cyclic test results of identical precast system obtained from literature. Differences between the MRSA and pushover approaches are presented and discussed. Performance levels of the structural system were subjected to three levels of design ground acceleration (0.05, 0.075 and 0.1 g) have been included.

1. Introduction

The government of Malaysia has been strongly encouraging the use of Industrialized Building System (IBS) in the construction industry especially for large projects. The precast concrete method and structural steel are the two major components of IBS. However, the level of acceptance of local contractors in using precast concrete construction is very low [15]. This calls for local precast suppliers to take the initiative to develop their own product lines in order to respond to the call from the authorities. Among them are the HC Precast System (HCPS), which consists of structural wall panels prefabricated off-site. The wall panels are joined at site through wet concreting along the vertical joints (Fig. 1). Instead of using conventional timber formwork for site concreting, the modular mould [15] was invented by the system supplier to improve the reusability as well as to speed up the construction process.

Fig. 2 shows the force transfer mechanism along the vertical interface between the wall panel and the column. While vertical force (i.e. design load of the structure) is mainly resisted by the shear keys, resistance against lateral force depends

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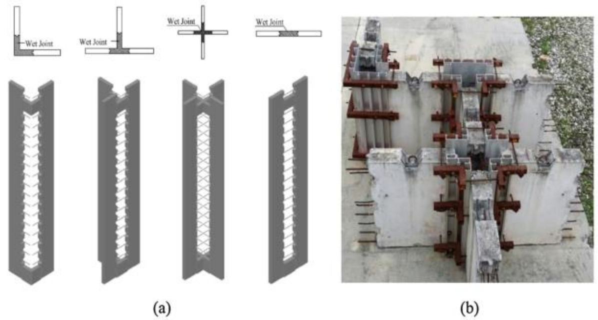

Fig. 1. (a) Commonly used joints in HCPS configuration and (b) Reusable modular moulds for wet joint concreting.

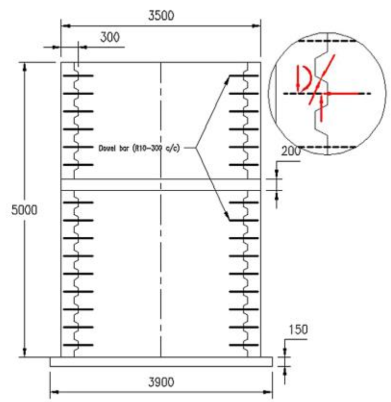

Fig. 2. Configuration of shear keys and dowel bars along interface as well as the internal force transfer mechanism at the connections.

on the dowel bars between the two concrete components. Thus, the two governing damage models of the interface can be either shear or crushing of concrete at the shear keys, or pullout of dowel bars.

Although severe seismic incidents are rarely reported in Malaysia, the occurrence of far field seismic effects from the Sumatra earthquakes in recent years has led to awareness by the government to initiate seismic designs in practice. With such effort, the Institute of Engineers of Malaysia (IEM) formed a Technical Committee (TC) concentrating on the formulation of seismic design codes suitable for the community of Malaysia based on Eurocode 8 (EC8) [7].

The early development of seismic hazard maps for Malaysia began in the early 2000s. [1] proposed the deterministic seismic hazard map for Peninsular and East Malaysia for the first time. Different seismic zonation maps were later proposed by [12] using probabilistic seismic hazard analysis. The seismic hazard maps that were developed by [1] and [12] suggested design ground acceleration of 0.1g to be used for a return period of 475 years. Fig. 3(a) and (b) shows the seismic hazard map for Peninsular Malaysia and Borneo respectively. Although these maps have been recommended to the government of Malaysia and have been used in some of the projects, they have yet to be made the official seismic design guidelines for the country. There are several extended works carried out by several other researchers over the years [14,3]. [13] proposed bedrock acceleration of 16.5 and 23.4 gal (1000 gal = 1g) for 10% and 2% probability in 50 years for Kuala Lumpur. The Technical Committee (TC) of seismic code comprising of mostly practicing engineers regarding the proposed design ground acceleration is still of concern whether such level of acceleration will cause major changes to current conventional design of structures. Hence, the TC has proposed 0.05g to be used as the design ground acceleration for normal building structure [8]. Meanwhile,

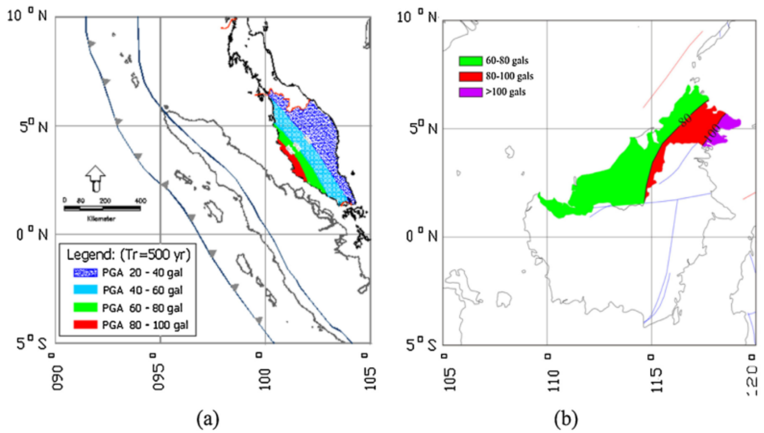

Fig. 3. Seismic hazard maps for (a) Peninsular Malaysia and (b) Borneo [1].

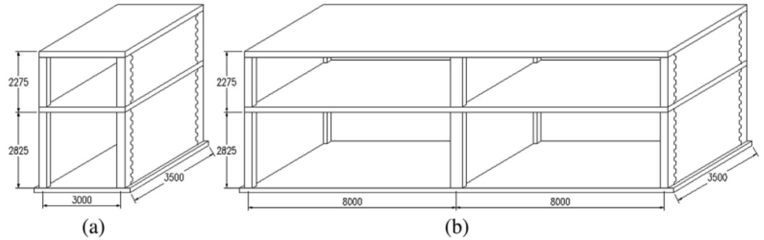

Fig. 4. (a) HCPS-VL1 and (b) HCPS-VL2 and HCPS-VL3.

the Public Works Department of Malaysia (JKR) has taken the average between these two values (0.075g) in the designing of important structures such as highway bridges and dams [9].

For this study, these design ground accelerations are still considered informal design values since there is no agreement on a fixed value as the official design parameter for the country. Regardless of the development of EC8 particularly for the National Annex of Malaysia, the implementation of these European design standards will definitely create an impact to both existing structures as well as new buildings. Existing buildings need to be checked whether retrofitting is necessary and new structures have to be designed to resist the lateral loads from seismic ground motion stated in the codes. Therefore, this paper examines the effect of using these different values of ground acceleration for seismic analysis and design of HCPS using both pushover and modal analyses using FE model that has been validated against laboratory test results obtained from literature. In addition, should the system be unable to meet the expected performance under earthquake loading; the current structural design of the HCPS needs to be reviewed to avoid massive retrofitting works in future.

2. HC precast system (HCPS)

The HCPS is used in the construction of residential housing and commercial shop houses in the country. The system supplier has limited the market size to only these building structures in order to make it possible for standardization of the wall panel dimensions. Nevertheless, the length of bays supported by these wall panels may vary from a short (approximately 3 m) to a long span of 8 m depending on thickness of the precast slab spanning between them as shown in Fig. 4. The structural configurations chosen for the case study are presented here.

Table 1
Three different configurations of HCPS with different imposed vertical loads.

	HCPS-VL1	HCPS-VL2	HCPS-VL3
Wall type	Exterior	Interior	Interior
Slab length	3.5 m	8.0 m	8.0 m
Nos. of story	2	2	2
Live load	1.5 kN/m^2	1.5 kN/m^2	4.0 kN/m^2
Total weight	265 kN	995 kN	1475 kN

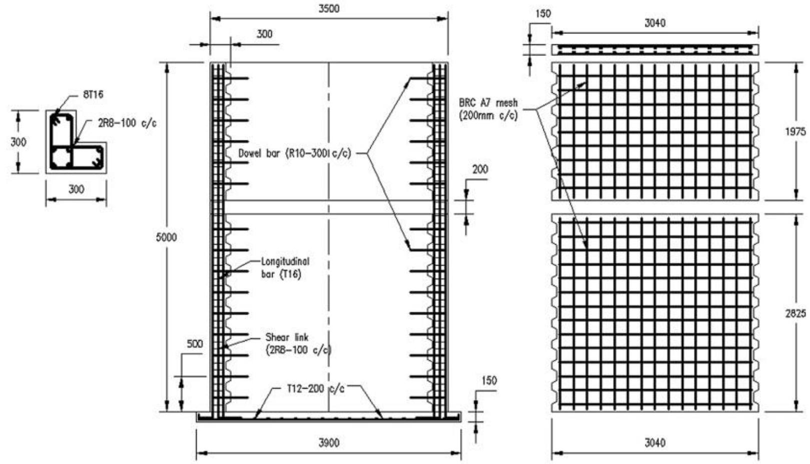

Fig. 5. Structural configurations of HCPS.

Three types of vertical loading were considered in the seismic analyses. The dead load (DL) was taken from the self-weight of the structural elements themselves while live load (LL) was obtained from BS 6399-1 [5]. Although the building codes have been replaced by Eurocode, British Standards are still widely used in the country during the current transition period. Loading from non-structural element was excluded in the model since it was not included in the quasi-static cyclic test in the first place [11]. HCPS-VL1 consisted of possible minimum loading that could be imposed on the structure while HCPS-VL3 comprised the probable maximum loading. The intermediate vertical loading, which represented typical weight carried by many shop house structural layouts, was denoted by HCPS-VL2. The maximum intensity of distributed load for LL was taken as 4.0 kN/m^2 due to the wide range of possibility of commercial shop lot usage while the minimum one was 1.5 kN/m^2. External wall refers to the perimeter wall (Fig. 4(a)) while the interior wall refers to the middle wall in Fig. 4(b). The three types of HCPS loading are listed in Table 1. Details of reinforcement and structural configuration of HCPS are shown in Fig. 5.Concrete grade C30 was used for all concrete elements. The structure was designed according to BS 8110-1 [6] without earthquake loading. However, the notional load due to geometry imperfection was considered in the design by taking into consideration lateral load of 1.5% storey mass.

In a finite element (FE) analysis, the column was modeled as a frame element with possible plastic hinges at critical regions as stated in FEMA 356 [10]. Meanwhile, the wall panels were represented by nonlinear shell elements. The most significant part of the modeling was the interface between wall and column. The shear keys have been represented by a series of rotational springs fully restrained in all six degree-of-freedoms. Dowel bars were modeled as nonlinear link elements taking into consideration strength degradation due to pullout. The ultimate anchorage resistance (V_b) of dowel bars was estimated using Eq. (1) [6]. Considering that the castellated joint was unreinforced, ultimate allowable shear stress of 1.3 MPa as recommended in BS8110-1 was used to determine the shear key resistance. Detail FE model of the entire interface was complex and is presented briefly when dealing with pushover analysis in this paper. For further reading, refer to [16]. However, validation of the proposed FEM against the hysteresis loops from laboratory test data [11] is shown in Fig. 6. The proposed FE model shows good agreement with the hysteresis loops obtained from the quasi-static cyclic test. It should be noted that the nonlinear behaviour is only activated in the pushover analysis. In the linear analysis, the FE

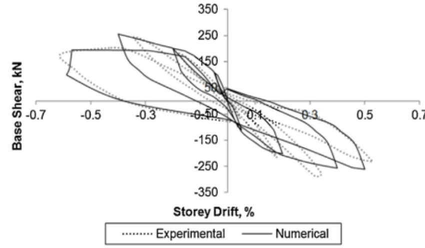

Fig. 6. Hysteresis loops of HCPS from experimental and numerical modeling.

Table 2
Recommended seismic analysis of structure in EC8.

	Linear	Nonlinear
Static	Equivalent static analysis	Pushover analysis
Dynamic	Modal response spectrum analysis	Time history analysis

Table 3
Mode shape, period T, and modal participation factor M_X of HCPS-VL1, HCPS-VL2 and HCPS-VL3.

Mode (n)	HCPS-VL1		HCPS-VL2		HCPS-VL3	
	T_n (s)	M_{Xn}	T_n (s)	M_{Xn}	T_n (s)	M_{Xn}
1	0.069	0.890	0.109	0.920	0.162	0.920
2	0.028	0.110	0.038	0.079	0.053	0.000
3	0.026	0.000	0.037	0.000	0.050	0.067
4	0.016	0.000	0.021	0.001	0.026	0.004
5	0.012	0.000	0.017	0.000	0.025	0.000
6	0.009	0.000	0.013	0.000	0.017	0.000
7	0.008	0.000	0.012	0.001	0.016	0.002
8	0.006	0.000	0.010	0.000	0.013	0.000
9	0.006	0.000	0.009	0.000	0.012	0.000
10	0.005	0.000	0.008	0.000	0.011	0.000
$\sum Sum$		1.000		1.000		0.993

analysis utilized only the initial (linear) stiffness of each element. Initial stiffness of the dowel bar was estimated based on the deformation data in [4].

$$V_b = 0.6F_b \tan\alpha_f \tag{1}$$

Where F_b = anchorage values of reinforcement. and α_f = internal friction coefficient

3. Linear analysis

In most seismic analyses using the EC8 approach, a simplified method using linear analysis are permitted to analyze structure that fulfills certain geometry requirements. There are four main different seismic analysis methods recommended in the EC8 as listed in Table 2.

Unlike the American codes [2] in which preferences are given to the equivalent static analysis or sometimes termed as lateral static analysis, EC8 regards the modal response spectrum method as the source of method. The modal response spectrum method can be used for analysis of buildings without limitation meanwhile the lateral static method requires certain corrections due to the contribution of effective modal mass for structures of different heights, estimation of structural fundamental period T_1, etc. Therefore, this study adopted the modal response spectrum method in carrying out the linear analysis instead of the common equivalent static procedure. Before performing the modal response spectrum analysis, a modal analysis was carried out to determine the number of mode shape to be included. The modal analysis results of HCPS are shown in Table 3.

The first two mode shapes for HCPS-VL1, VL2, and VL3 are illustrated in Fig. 7. It was noted that as the vertical load imposed on HCPS increased, while the relative story displacement of the structure for both first and second mode decreased. The level of decrease of the second mode was drastic compared to the first mode.

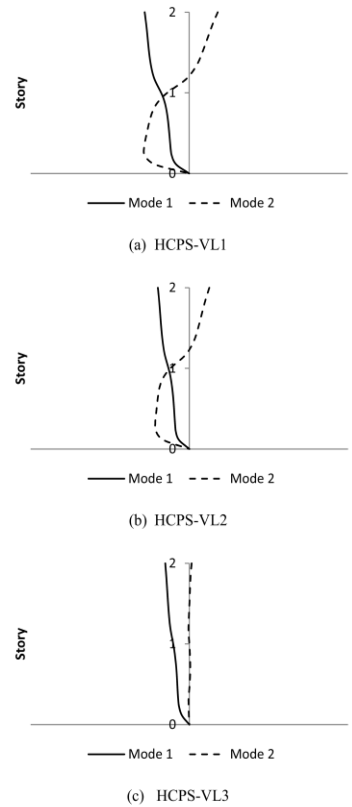

Fig. 7. First two mode shapes of HCPS from modal analysis.

The numbers of mode shape to be included in the combination of modal responses were taken to be ten (10). Based on the EC8 criterion, the numbers of N mode need to be considered to ensure that the effective modal mass (M_X). participation accumulated to more than 90% of the total mass of the structure along the direction of interested seismic actions.

Using modal parameters mentioned in previous paragraphs, t Modal Response Spectrum Analyses (MRSA) were performed for the three HCPS configurations using the Type 1 horizontal response spectrum of the Eurocode 8. Design ground aation $\left(a_g\right)$. of 0.05 g, 0.075 g and 0.1 g were assigned to the linear horizontal acceleration spectrum (S_e). of five different soil conditions (ranging from hard rock S_A to soft soil S_E) in the response spectrum analysis. It is worthwhile to mention that behaviour factor of 1 is used in both the linear MRSA and nonlinear pushover instead of the EC8 proposed value of 1.5. The rationale behind this is to avoid cracks along the wall-column interface which is difficult to repair. The horizontal elastic response spectra (S_e). for design ground acceleration $a_g = 0.1$ g of Peninsular Malaysia are shown in Fig. 8.

Results of peak values of seismic action effect determined from the MRSA have been divided into three groups as follows:

(a) Global effect: base shears,
(b) Local effect: member internal forces, and
(c) Intermediate effect: interstorey drifts.

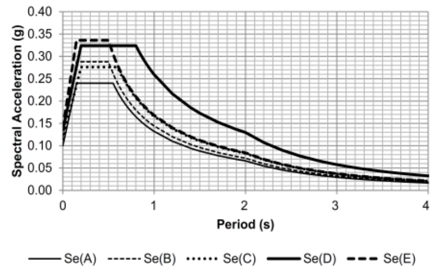

Fig. 8. Spectral acceleration of Peninsular Malaysia (a_g) = 0.1 g for different soil classifications.

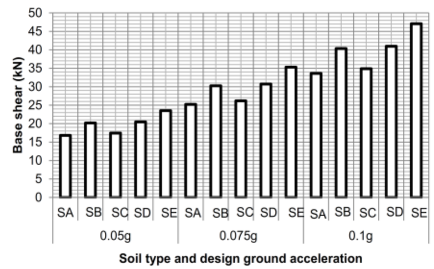

Fig. 9. Base shear of HCPS-VL1 for different design ground accelerations and soil classification from MRSA.

Table 4
Dowel reaction of HCPS-VL1 within pullout capacity.

a_g(g)	Soil Type	Dowel Element	
		Force Ratio	Status
0.05	SA	0.723	[a]N.D.P.
0.05	SB	0.867	N.D.P.
0.05	SC	0.750	N.D.P.
0.05	SD	0.881	N.D.P.

[a] *N.D.P. = no dowel pullout.*

The base shear values obtained for HCPS-VL1 ranged from 15 to 50 kN as shown in Fig. 9 for ground excitations from 0.05 g, 0.075 g to 0.1 g in all five soil classifications. These base shear values were all within the linear response of HCPS since the expected yield point of HCPS was 95 kN (from the quasi-static cyclic test data). From the graphs plotted in Fig. 9, the distribution pattern of base shear for HCPS-VL1 increased from soil type A to E, when the design ground acceleration increased. However, under the same peak ground acceleration (PGA) group, the values of lateral base shear between soil type A and C were close to each other. The same pattern was noted between soil type B and D. The base shear excited at site containing soil type E marked the highest value within the same level of each PGA group. This was attributed by the fact that although the peak and shape of the design spectra differed among soil types, similarity occurred within the short period region between the mentioned soil category. Since the MRSA employed a combination of peak responses of each vibration mode, obtaining approximate base shear values between different soil types was possible, as HCPS-VL1 happened to be in the short period region ($<T_B$) due to low mass-to-stiffness ratio.

Meanwhile, the internal forces within column members and internal stresses of wall panels were all within the force ratio of 1.0. The force ratio is calculated the ratio between internal force demand (effect) over design capacity (resistance) of the structural element. In other words, no plastic hinge formation occurred at the column members or crushing of concrete walls. Nevertheless, the response of dowel forces was interesting. The dowel action was within its maximum pullout capacity of 13 kN under the four seismic loading cases as listed in Table 4. In other words, any MRSA greater than 0.05g would yield internal dowel force that require either enlargement of rebar diameter or lengthier anchorage in order to keep the connection

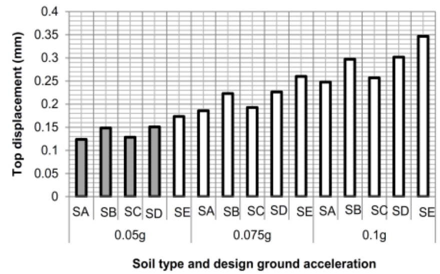

Fig. 10. Roof displacement of HCPS-VL1 obtained from MRSA.

Table 5
Force ratio of column, wall, and dowel bar for HCPS-VL3 obtained from MRSA.

a_g(g)	Soil Type	Column Element		Wall Element		Dowel Element	
		Force Ratio	Status	Stress Ratio	Status	Force Ratio	Status
0.05	SA	<1	Linear	0.003	[a]N.C.C.W	4.111	[b]D.P.
0.05	SB	<1	Linear	0.004	N.C.C.W	4.934	D.P.
0.05	SC	<1	Linear	0.003	N.C.C.W	4.211	D.P.
0.05	SD	<1	Linear	0.004	N.C.C.W	4.943	D.P.
0.05	SE	<1	Linear	0.004	N.C.C.W	5.756	D.P.
0.075	SA	<1	Linear	0.004	N.C.C.W	6.167	D.P.
0.075	SB	<1	Linear	0.005	N.C.C.W	7.400	D.P.
0.075	SC	<1	Linear	0.004	N.C.C.W	6.317	D.P.
0.075	SD	<1	Linear	0.005	N.C.C.W	7.415	D.P.
0.075	SE	<1	Linear	0.006	N.C.C.W	8.634	D.P.
0.1	SA	<1	Linear	0.006	N.C.C.W	8.223	D.P.
0.1	SB	<1	Linear	0.007	N.C.C.W	9.867	D.P.
0.1	SC	<1	Linear	0.006	N.C.C.W	8.422	D.P.
0.1	SD	<1	Linear	0.007	N.C.C.W	9.887	D.P.
0.1	SE	1	Overstressed	0.008	N.C.C.W	11.512	D.P.

[a]N.C.C.W. = no crushing of concrete wall, [b]D.P. = dowel pullout.

intact or rigid. The minimum internal force-over-pullout capacity ratio ranged from 1.1 for a_g =0.05 g to 2.1 obtained from a_g =;0.1 g, indicating that pullout of dowel bars would occur at the wall-to-column interface.

The results of linear roof displacement are shown in Fig. 10. While these results are deemed valid only for soil condition class A to D (shaded in grey in the figure) under 0.05*g* design ground acceleration, the rest of the displacement require nonlinear analysis since the dowel bars were expected to yield into its local nonlinear force-deformation region due to pullout. Since MRSA is always in linear mode, the distribution pattern of peak structural displacements of HCPS at roof level corresponded to the amount of base shear resisted by the structure. In the linear response range according to MRSA analyses, the peak displacements of HCPS at roof level ranged between 0.13 mm, which corresponded to 0.003% drift to 0.35 mm or 0.07% drift.

In HCPS-VL2 configuration, the internal force demand in column and wall element was within linear limit but similar to HCPS-VL3 (Table 5) in terms of dowel bar reaction, all analyses revealed large force ratio. The largest force-over-capacity ratio obtained was 11.5 in HCPS-VL3 under design ground acceleration 0.1*g* for soil type E.

4. Pushover analysis

A pushover analysis for HCPS-VL1, HCPS-VL2, and HCPS-VL3 was carried out using Single Point Loading (SPL) configuration because it produced the most conservative capacity curve compared to the other Uniform Distributed Loading (UDL), Modal Distributed Loading (MDL) and Triangular Distributed Loading (TDL) configurations [16]. Location of the applied lateral load in the SPL pushover analysis (represented by actuator in the laboratory test setup by [11] is shown in Fig. 11. The nonlinear pushover analysis revealed that all three HCPS configurations performed within Immediate Occupancy (IO) structural performance level (S-1) according to [10] in all demand spectra given. The pre-standard, FEMA 356 [10] has classified four discrete structural performance levels for building exposed to seismic force. Immediate Occupancy (IO) means that the building is safe to be occupied after the earthquake in which no structural stiffness and strength degradation have occurred. However, although the structural response in particular HCPS-VL3 remained within its elastic domain under 0.05 g design

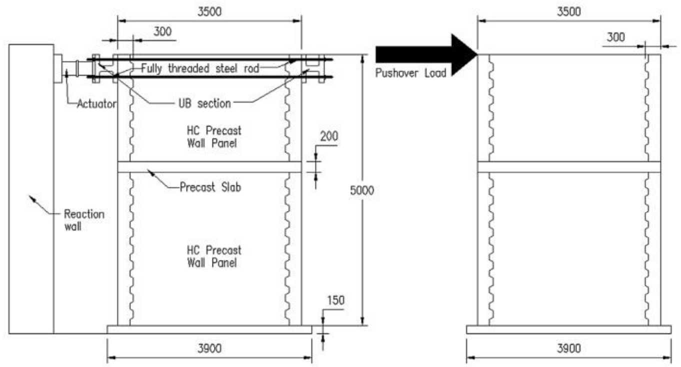

Fig. 11. Applied lateral loading in pushover analysis (represented by actuator in laboratory test setup).

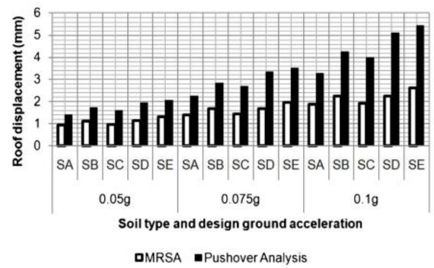

Fig. 12. Roof displacement demand of HCPS-VL3 from pushover and MRSA.

ground acceleration, the structure was in its over-strength region in a_g of 0.075g and 0.1 g despite possessing higher vertical loading close to its ultimate design strength.

In the nonlinear model, the shear key protruded along the height of column was represented by a rotational spring element with highly rigid moment-rotation behaviour. Next, the dowel action was assumed to be responsible for resisting all tensile force between the wall and column. A translational nonlinear link was assigned to represent each dowel action. The maximum anchorage resistance of a dowel bar was estimated using Eq. (1). The plastic behaviour of dowel reaction was represented by means of the force-deformation relationships based on the bi-linear model. Initial stiffness of the dowel bar was estimated based on the deformation data in [4]. Considering that upon reaching maximum pullout capacity, the dowel bar has very minimal residual strength to resist further tensile force; a sudden drop of strength (130 kN/mm) was assigned as the post-yield stiffness. Another nonlinear link element was also introduced to represent the shear key contact surface or interface between the precast panel and column members. While this surface would purely be attributed to plain concrete, the weak tensile strength of the concrete was modeled by hook element and the compressive strength of concrete shear key included shear failure mechanism of the element.

The estimation of acceleration response, roof displacement demand, and base shear of HCPS-VL1 using both pushover analysis and MRSA did not reveal significant differences. The major reason is that the structure was still responding within its elastic domain. However, compared to the pushover analysis for HCPS-VL2 and HCPS-VL3, the MRSA underestimated seismic displacement demand by almost 50% (Fig. 12) while overestimating the base shear and acceleration response by 45% and 77% respectively. The discrepancy occurred when the structural response was beyond yielding point.

Interestingly, the internal stress demand of structural elements for HCPS obtained between MRSA and pushover analysis was very different. Comparisons between Tables 5 and 6 indicate that the pushover analysis revealed higher stress demand within the wall element and column while having lower dowel reaction. As MRSA treated the wall-to-column interface with higher stiffness compared to the nonlinear dowel reaction allowed in pushover analysis, higher force was concentrated

Table 6
Force ratio of column, wall and dowel bar for HCPS-VL3 obtained from pushover analysis.

a_g (g)	Soil Type	Column Element		Wall Element		Dowel Element	
		Force Ratio	Status	Stress Ratio	Status	Force Ratio	Status
0.05	SA	<1	Linear	0.074	[a]N.C.C.W	0.210	[b]N.D.P.
0.05	SB	<1	Linear	0.074	N.C.C.W	0.210	N.D.P.
0.05	SC	<1	Linear	0.074	N.C.C.W	0.210	N.D.P.
0.05	SD	<1	Linear	0.074	N.C.C.W	0.210	N.D.P.
0.05	SE	<1	Linear	0.074	N.C.C.W	0.196	N.D.P.
0.075	SA	<1	Linear	0.074	N.C.C.W	0.196	N.D.P.
0.075	SB	<1	Linear	0.152	N.C.C.W	0.748	*N.D.P.
0.075	SC	<1	Linear	0.152	N.C.C.W	0.748	*N.D.P.
0.075	SD	<1	Linear	0.152	N.C.C.W	0.748	*N.D.P.
0.075	SE	1	Overstressed	0.152	N.C.C.W	0.748	*N.D.P.
0.1	SA	1	Overstressed	0.122	N.C.C.W	0.208	N.D.P.
0.1	SB	<1	Linear	0.153	N.C.C.W	0.968	*N.D.P.
0.1	SC	<1	Linear	0.152	N.C.C.W	0.748	*N.D.P.
0.1	SD	<1	Linear	0.153	N.C.C.W	0.968	*N.D.P.
0.1	SE	1.25	Overstressed	0.168	N.C.C.W	0.968	*N.D.P.

[a]N.C.C.W. = no crushing of concrete wall, [b]N.D.P. = no dowel pullout.

within the dowel bars in MRSA. The linear MRSA was observed to over-estimate the dowel force demand exerted by the seismic action onto HCPS. Although the base shear values between the linear MRSA and nonlinear pushover analyses were relatively in tandem, the roof acceleration responses between the two differed vastly.

The reason is due the difference of wall-to-column interface mainly the dowel stiffness that was used in these two analyses. This only occurred when the seismic demand had exceeded those linear capacities of HCPS, such as those occurring in 0.1g for soil type E. In the linear analysis, the model only took into account the initial stiffness of dowel bar due to the procedure in linear procedure. The degradation of local joint was unable to be included in the analysis regardless of occurrence of any yielding. Meanwhile, the nonlinear pushover analysis was able to utilize the true force-deformation relationship established for the dowel action due to pullout, and failure in shear key elements. These local nonlinear effects caused additional energy dissipation of the HCPS and thus leading to a less stiff structure compared to the linear model. As a result, the roof acceleration responses became lower than the linear analyses. The same theory applies to the estimation regarding displacement.

5. Conclusion

MRSA and pushover analysis of the three vertical loading configurations of HCPS under Malaysia earthquake conditions were performed using response spectrum developed by [1] for the country. The study shows that MRSA overestimated the dowel reaction in HCPS due to the nature of analysis that combined all internal forces within the link elements in positive values. This treated all forces in tension acting along the dowel bars. Hence, dowel pullout was observed from MRSA even at very small design a_g (as low as 0.05*g* in soil classification class E).

The pushover analysis revealed that regardless of the level of design ground acceleration used, HCPS-VL1, HCPS-VL2 and HCPS-VL3 remained within Immediate Occupancy (IO) structural performance level according to [10]. Thus, retrofitting of completed buildings within the vertical loading used in this study is not required when the EC8 is to be implemented in Malaysia in the near future. However, further assessment is required if the structural configuration differs vastly from the one used in this study or when the geometrical aspects do not meet the requirements of EC8. In addition, the out-of-plane resistance and behaviour of the wall panel may be further investigated.

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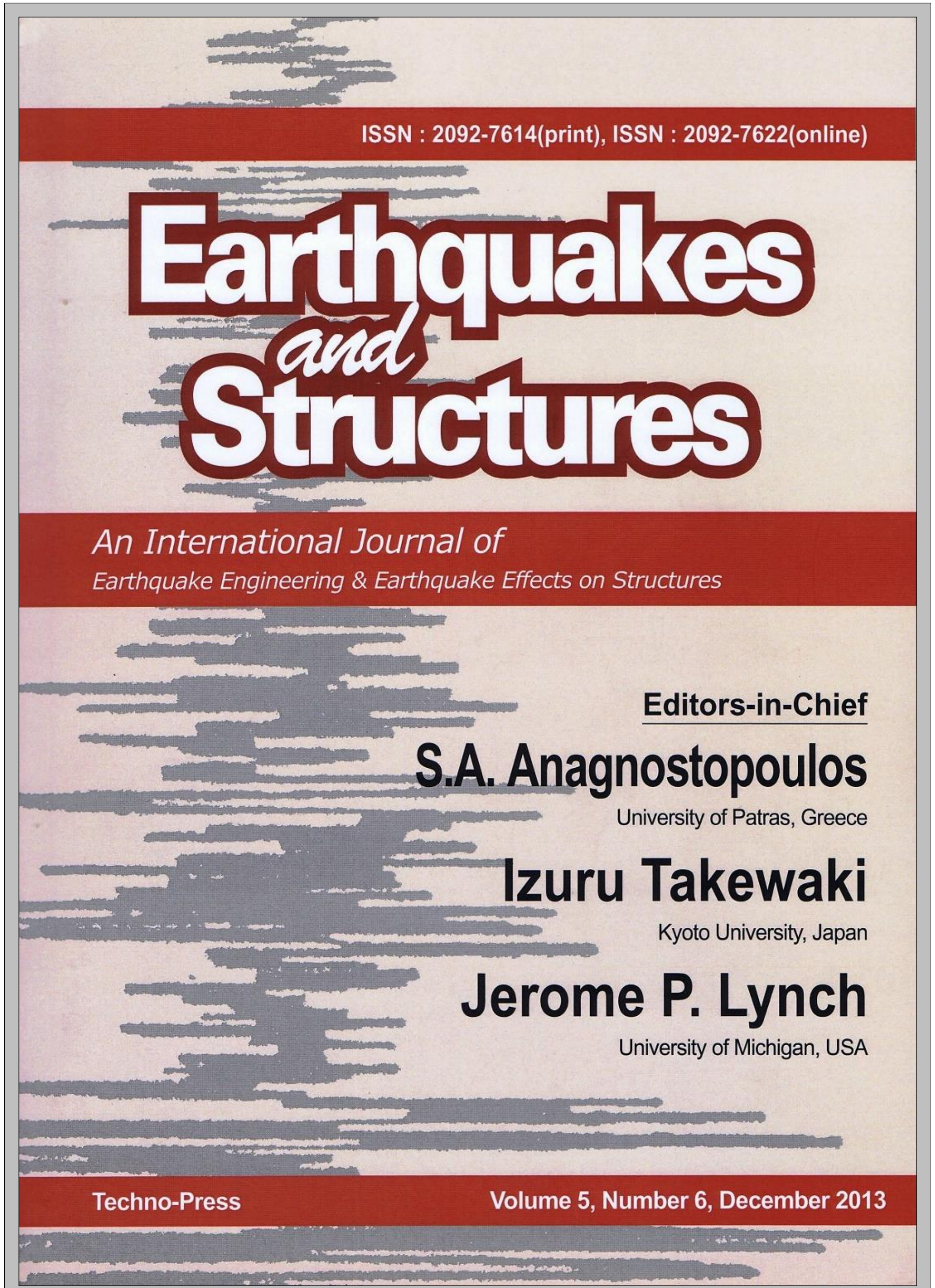
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Behaviour factor and displacement estimation of low-ductility precast wall system under seismic actions

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Abstract. This paper investigated the seismic behaviour of an innovated non-ductile precast concrete wall structural system; namely HC Precast System (HCPS). The system comprises load-bearing precast wall panels merely connected only to column at both ends. Such study is needed because there is limited research information available in design codes for such structure particularly in regions having low to moderate seismicity threats. Experimentally calibrated numerical model of the wall system was used to carry out nonlinear pushover analyses with various types of lateral loading patterns. Effects of laterally applied single point load (SPL), uniformly distributed load (UDL), modal distributed load (MDL) and triangular distributed load (TDL) onto global behaviour of HCPS were identified. Discussion was focused on structural performance such as ductility, deformability, and effective stiffness of the wall system. Thus, a new method for engineers to estimate the nonlinear deformation of HCPS through linear analysis was proposed.

Keywords: low ductility; displacement factor; seismic; precast concrete wall; pushover

1. Introduction

Benefits demonstrated by precast concrete building technique over the conventional cast in-situ method have been long proven in many large constructions over the world. This is clearly revealed by the widely applicable seismic design provisions for precast structures such as those contained in International Building Codes (International Code Council 2009) and Eurocode 8 (CEN 1998). Although the advancement of precast concrete industries is highly demonstrated among developed nations such as the United States and most of the European countries, its implementation among developing countries is reportedly low. Despite strong encouragement of local governments such as those of Malaysia, the level of acceptance of the precast technology is still reportedly low (Haron *et al*. 2005, Hassim *et al*. 2009).

Hence, it has become important for the private industry to initiate relevant researches onto prospective precast system that best suits the needs of local industry. Among them is the HC

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Fig. 2 Panels of HC precast wall system

of wall panel was calculated from the difference between the stresses at upper storey and lower storey of the wall level in consideration. The margin of shear stress (or the residual shear stress) was then resisted by the shear keys along the vertical interface. Such approach clearly, assumed that these shear stresses which occurred along the vertical interface between walls were perfectly perpendicular to the direction of lateral loading. In other words, these shear keys were under direct shear demand despite the direction of lateral loading might not always be constant.

Raths (1977) and Christiansen (1973) presented step-by-step analysis and design of precast concrete load-bearing wall panels for high-rise construction in Georgia (Seismic Zone 1). The structural system for the building consisted of precast bearing wall façade in the exterior and also interior precast frame elements. The lateral loads were then, assumed to be resisted by both the frame and load-bearing wall systems, separately. The natural period of the designed building was obtained by empirical formula, and seismic design force was determined using the equivalent static force method based on the Uniform Building Code (UBC). The load-bearing walls were treated as overlapping strut with stubs elements. The interfaces between wall-to-wall and also wall-to-frame were all assumed to be perfectly rigid. At vertical interface between adjacent wall panels, interface release was carried out to allow only vertical shear stress transfer along the wall height. Such analytical method once again ruled out the possibility of any other force demand occurring at these interface locations due to seismic force. Nevertheless, this was the best effort for analysis to be possible during then. Only two types of shear stresses were considered to occur within the building. First were the horizontal shear stresses which took place at each storey level. And secondly, the vertical shear stress due to flexural behaviour (bending caused tension and

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Shuttlelift, Sturgeon Bay, WI 54235-1976, USA

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Strusoft, 21774 Malmö, Sweden

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HC Precast System Sdn. Bhd., 41200 Klang, Selangor Darul Ehsan, Malaysia

Improved Discrete Precast Concrete Wall Panels and Modular Moulds for Wet Joints in Malaysia

Despite the long history of precast concrete construction, the level of acceptance of the precast wall panel system in Malaysia is still considerably low. One of the reasons is the impediment of dry connection usage due to the humid and wet tropical climate throughout the whole year season. As a result, wet joint is the remaining choice for the country. Contemporarily, most precast concrete manufacturers in the nation are still resorting to conventional timber-made formwork when it comes to temporary strutting works and formworks for joints concreting. This article discusses problems faced by the construction industry in Malaysia that currently lead to lower precast concrete utilization throughout the country. Next, an innovated modular formwork which can be easily assembled and dismantled for in-situ concreting purpose to be used with precast concrete wall panels is presented. The main objective of the paper is to introduce a more comprehensive 'precast concrete construction system' for wet jointed precast works particularly among most developing countries in the South East Asia region to improve the current 'precast concrete components' construction sequence. It is worth to mention that this article is written from the viewpoint of the Malaysian construction industry.

■ Patrick Tiong Liq Yee, Professor Dr Azlan Adnan, Universiti Teknologi Malaysia;
Fadzil Ahmad, HC Precast System Sdn. Bhd. ■

Besides wet weather condition that impedes usage of dry joint, another setback is caused by lacking of good interaction between the M & E (mechanical and electrical) system provider and the precast manufacturer. In conventional brickworks construction method, the M & E conduits are finalized on site after the concrete skeletal frame is completed. Using of precast panel construction requires the M & E layout to be confirmed right before the fabrication of the precast panels. Consequently, the traditional masonry infill panels are still widely preferred by the nation's building industry.

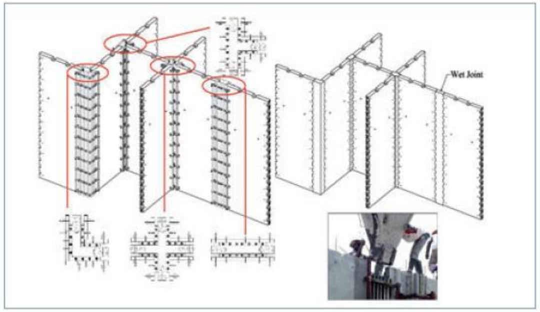

Fig. 2: Modular moulds for wet joints

Higher construction cost faced in utilizing precast concrete components has worsened the matter. Since precast method is a relatively new construction trend among the nation, the end products are often sold at higher price in order to level out with the initial investment capital. Besides that, regularly requirement for product customization which still hinders efficient mass production in Malaysia also contributes to an expensive precast system in this region. Unless the precast construction system can really demonstrate strong benefits compared to the existing cast in-situ method, its growth will not be too promising among local builders.

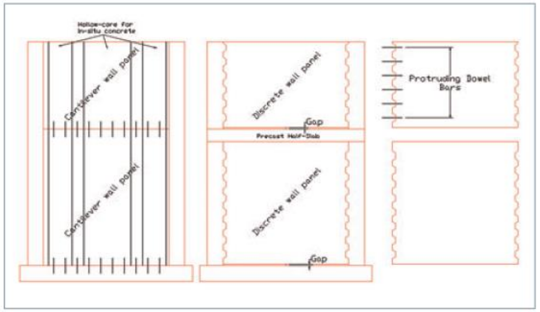

Fig. 1: Cantilever (left) and discrete precast wall panel (right)

The cantilever precast wall (Figure 1-a) is a type of wet jointed precast wall structure which required considerably large amount of cast in-situ concrete and steel works. In order to enhance the applicability of precast panel in the country, the company (HC Precast System Sdn. Bhd.) innovated a new precast wall panel system by improving some characteristics of the typical cantilever wall system. Major modifications include the development of column-wall interaction through protruding dowel bars

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Fadzil Ahmad is currently the Managing Director of HC Precast System Sdn. Bhd., who is also one of the system owners. He holds a Diploma in Quantity Surveying from Universiti Teknologi Malaysia and also a Master Degree in Business Administration (MBA) from Keele University, United Kingdom. His biggest vision is to promote and enhance awareness of the construction industry regarding precast concrete utilization.

and shear keys alongside the wall vertically, as well as discontinuity of the wall panel in the horizontal direction. The new system (Figure 1-b) is tested and evaluated, and some improvements to the precast technology are presented in this article.

The precast panel acts as beam and wall at the same time. It carries the loading from the slab and transfers the force to the connecting columns through the shear keys element. The wall panel is designed only to carry vertical gravity load, with only mesh

Fig. 3: Example of '+' shaped modular mould and installation

Fig. 4: Double groove lines for waterproofing

Fig. 5: Waterproofing test. Top floor with water (left). Bottom floor withour leakage (right)

reinforcements provided within the panel. However, laboratory test of the new system under lateral cyclic load reveals that the plain panel has increased the lateral capacity of the column by nearly 300%, compared to beam-column frame structure. On 18th August 2011, a double storey one-third smaller scale model of the system which comprises precast panels and half-slab was tested for earthquake resistance for 8 series of seismic ground accelerations. Earthquake records from New Zealand, Turkey, Iran, Italy, Taiwan, Japan, California, and artificial generated time history for Malaysia were among the selected ground motion records. The smaller scale precast model remained in the elastic region throughout the test, with no visible cracks occurred.

Precast wall panel construction of typical residential houses requires at least 4 types of vertical wet joints, as shown in Figure 2 to connect the wall panels together in the vertical direction. The 'L' shaped connector joints two perpendicular walls together, while the '-' connects two side-by-side panels. The 'T' joint is required when 3 adjacent walls are connected and the '+' shaped connection is used at cross joint between 4 panels. These wet joints require sufficient formwork or mould for in-situ concrete casting. A series of modular moulds which is not only reusable, but also easily assembled, dismantled and even adjustable to any required sizing to act as temporary formwork for the concreting work is also invented (Figure 2 and Figure 3) by the company. The patented modular moulds enhance faster erection procedures of the precast wall system. Through prefixed holes alongside the precast wall panels vertically, the moulds are easily tightened in position by a specifically engineered interlocking mechanism.

Small voids within the wall panels for mechanical and electrical system conduits are easily prefabricated due to the relatively lesser amount of reinforcement within the panel. Consequently, the need of customizing the panel design to suit the M & E is avoided. With the layout of servicing lines readily proposed by the precaster, the finalizing works for M & E consultants become much easier and faster.

The system decreases the need of beam elements. The wet joint construction of precast concrete often requires a proper waterproof sealant or technique. This proposed wall panel come with double grooving lines along every edge (Figure 4). The groove lines are expected to develop better bonding between the precast panel and in-situ concrete, thus making it harder for water to penetrate through the bonding surface. The system has been put to test by exposing to outdoor weather for 4 years (Figure 5). The result indicates that the groove lines prevent effectively any water leakage possibility. These groove lines are created at the wall panel casting yard by utilizing particularly engineered moulds.

The proposed precast wall panel system has been successfully used together with the modular moulds in the construction of more than a thousand units of single and double-storey residential housing buildings, and also for a 5-storey commercial shop project. The proposed modular moulds serve as fast assembling and dismantling supporting formwork for wet joint concreting. The ability for recycling usage lowers formwork costs. The wall panel system increases the lateral load carrying capacity of the supporting columns almost by the factor three.

The future of construction industry is relying on the current development of building techniques. Lesser material wastage, shorter construction time and lower labors' requirements are some of the benefits demonstrated by precast concrete construction. This directly leads to sustainable and greener building method. It is hoped this article will improve further the practicality and application of wet jointed precast construction technique among fellow precasters in the South East Asia region.

FURTHER INFORMATION

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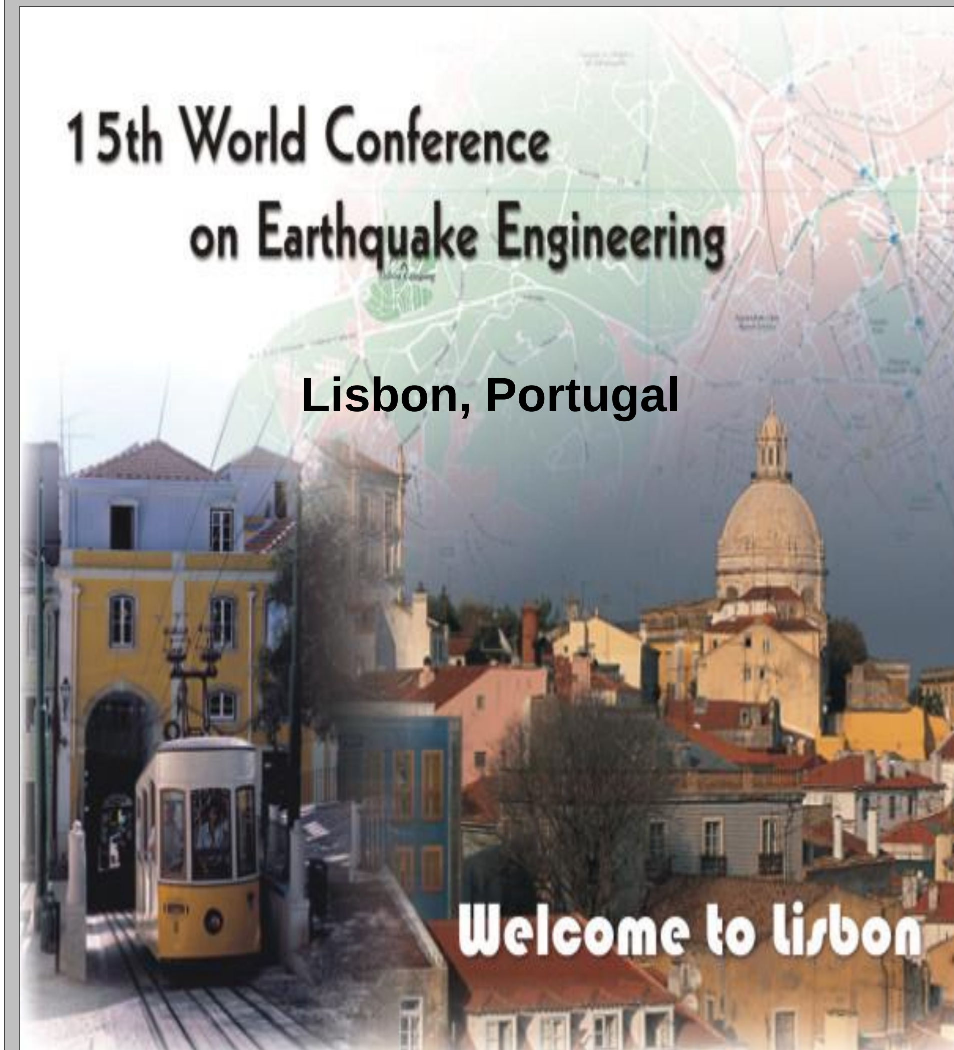

Lisbon, Portugal

Lisbon, Portugal

Ductility Factor of RC Building Frames with Different Infill Wall Configurations under Low Intensity Far Field Earthquake Effects

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INTRODUCTION

- It is not a well accepted practice for engineers to incorporate seismic analysis & design in Malaysian building industry.
- Introducing structural ductility-based design criteria into structural plan is less favored by practitioners.
- The main reason is the relatively awkward geographical location of Malaysia in low to moderate seismicity region.
- Therefore, it becomes important to understand the behavior of non-earthquake-resistance-design structure under such seismic threats.

STUDY AIMS

1. **The country is moving towards precast concrete construction due to strong encouragement from local government.**
2. **This study investigates the ductility behavior of an industry-innovated precast wall system (namely HCPS – HC Precast Wall System) under local soil demand spectrums.**
3. **Parameters studied: Effect of different wall location and vertical load onto HCPS ductility.**

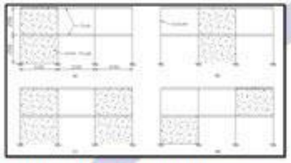
Background

Structural Model (HCPS)

METHODOLOGY

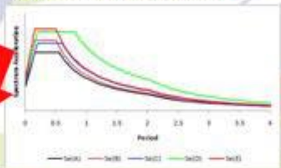
Wall Location

Four different wall locations were studied for two-storey, three-bay building system



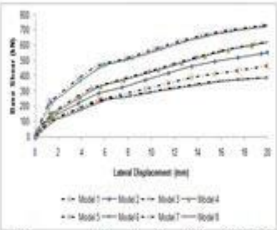
Loading Condition

- Two types of vertical loading:
  - Serviceability limit state (SLS)
    Represented by odd numbers (Model 1, 3, 5 & 7)
  - Ultimate limit state (ULS)
    Represented by even numbers (Model 2, 4, 6 & 8)
- Demand spectrums (Local)



RESULTS

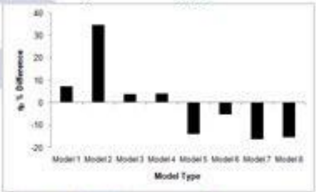
Pushover Analysis



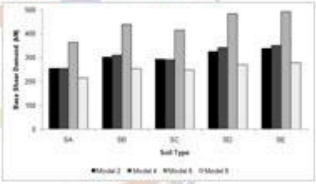
Difference of vertical loading (SLS & ULS) was only observed to affect ductility response of Model 1 & 2

RESULTS CONT'D

Ductility Factor (q_0)



Base Shear Demand



CONCLUSIONS

Location & configuration of HCPS very much affected the global ductility response of the system.

However, effect of SLS & ULS vertical loading was insignificant.

Ductility concept underestimates DCL structure in resisting low to moderate seismic intensity.

CONTACT US

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Shake Table Test of 1:3 Scaled HCPS Precast Wall System Journal by Nanyang Technical University

Shake table test of 1:3 scaled HCPS precast concrete wall system

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ABSTRACT: (10 pt)
Shake table test is probably one of the most preferred methods in dynamic testing of structures in earthquake engineering. However, due to the constraints imposed by size and payload limitation of the shake table facility, the test models are often smaller than their prototype structures. Although existing similitude theory allows for proper scaling of test models, it is sometimes impossible to strictly follow every single detail when it comes to practicality in construction of the scaled-down models. This paper investigates the shake table test characteristic of a distorted 1:3, two-storey precast concrete wall structure subjected to eight ground motion records covering both far-and-near field earthquakes. Verified FE models of the test structure were used to compare with the full-scale prototype building using the same group of earthquake records. This study revealed significance agreement between the distorted scaled-down model and prototype building marginal difference below 20 %.

Keywords: shake table test, precast wall, scale down, distorted model

1. INTRODUCTION

In earthquake engineering, one of the most important branches is shake table testing to investigate the dynamic behaviour of interested structures using earthquake records. Due to the limitations of shake table facility such as restraint in payload, or restriction of lab access and space constraint, the test structures need to be scaled down in most of the times. Scaling down of test model is not something new (Saito, 2008; Sunaryati, 2008 and Tiong, 2014). It can be done using the similitude theory derived from Buckingham theorem (Dove and Bernett, 1986). Nevertheless, it is almost impossible to appropriately assign the exact scaling factor in all aspects of the structural as well as dynamic components of the test model which will be presented in this paper. Hence, this study investigated the shake table test of a 1:3 scale precast wall building system invented and patented by a local precast manufacturer in Malaysia, namely HC Precast System Private Limited. The system is in line with the effort of Malaysian government to promote and enhance the usage of Industrialized Building System throughout the country to replace the existing conventional cast in-situ method. In addition to the shake table test, this study also compared the performance of the 1:3 scale test specimen to the FE analysis result of actual size prototype structure with the same structural configuration.

2. PROTOTYPE MODEL

Prototype model of the two-storey precast wall building, namely the HCPS-P is shown in Figure 1.1. The two-storey structure was made of two structural precast wall panels on both sides at each floor, connected by two cast in-situ concrete beams between the panels. These walls and beams supported concrete slab on top of them only in the middle storey. The total weight of HCPS-P was around 27 tons, with the dimension spanning at 3.5 m along wall side, 3 m in the opposite direction and a total of 5 m in height.

Figure 1.1. Full scale HCPS model for lateral cyclic loading test (Tiong *et al*, 2013)

The structure was designed in accordance to British Standards BS8110:1997-Part 1 (British Standards Institution, 1997), without any seismic loading. Therefore, the HCPS was only designed for normal gravity loading. After all, constructing a properly seismically designed structure in Malaysia will only aggravate the level of acceptance of IBS in Malaysia, which is already low (Hassim et al, 2009). Structural detailing of the can be found in Figure 1.2 and Table 1.1.

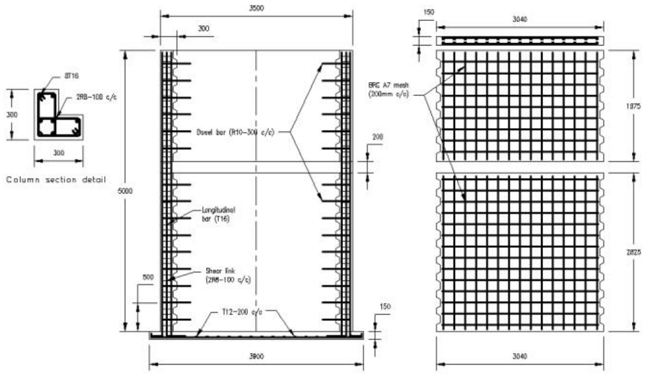

Figure 1.2. Structural detailing of HCPS test model (Tiong *et al*, 2013)

Table 1.1. Structural details of the full scale HCPS test model (Tiong *et al*, 2013)

Element	Section (mm)	Longitudinal Rebar (mm)	Shear Link (mm)
Wall (upper storey)	150 (t) x 3350 (w) x 1975 (h)	2 layers of BRC A7	Not provided
Column (upper storey)	300 (t) x 300 (w) x 1975 (h)	8T16	2R8-100 c/c
Beam (upper storey)	150 (t) x 450 (d) x 3000 (l)	2T12 (T) 2T12 (B)	R8-150 c/c
Wall (lower storey)	150 (t) x 3350 (w) x 2825 (h)	2 layers of BRC A7	Not provided
Column (lower storey)	300 (t) x 300 (w) x 2825 (h)	8T16	2R8-100 c/c
Beam (lower storey)	150 (t) x 300 (d) x 3000 (l)	2T12 (T) 2T12 (B)	R8-150 c/c
Concrete	Compressive strength = 30 N/mm^2		

3. 1:3 SCALED TEST SPECIMEN

Shake table test of the structure was performed at the Laboratory of Structures and Materials, Faculty of Civil Engineering of Universiti Teknologi Malaysia. It is one of the pioneer tests of its kind to be performed in the country. The term test specimen is used throughout the paper to represent scaled-down test model to avoid confusion with the full-size prototype test model. In order to emulate the actual precast construction quality and sequence of erection, these test specimens were produced at the precast manufacturing plant of HC Precast System Sdn. Bhd., located in Hulu Selangor, Malaysia. Two limitations imposed by the shake table facility and laboratory access had called for the need to scale down the specimen (see Table 3.1).

Table 3.1. Physical and performance limitation of shake table facility

No.	Parameter	Actual	Maximum Allowable	Remarks
1	Size	3 x 3.5 x 5 m	1.3 x 1.3 x 3.5 m	Scaling required
2	Weight	27,000 kg	1,000 kg	Scaling required

Dove and Bernett (1986) have developed series of scaling laws for structural responses under dynamic excitations. For reinforced concrete structures, scaling of material properties, included in constitutive similitude is not preferred. Therefore, the strength of concrete for the shake table specimen had to remain the same as those used for the quasi-static testing prototype. Scaling factors developed based on geometrical similitude factor (N_H) for mass (N_M), ground input acceleration (N_A), time domain (N_T), force (N_F), and constitutive material property (N_E) are listed in Table 3.2.

Table 3.2. Proposed scale factors for three different options (Dove and Bernett, 1986)

	Scale Factors				
Case No.	N_M	N_A	N_T	N_F	N_E
1	N_H^3	N_H^{-1}	N_H	N_H^2	1
2	N_H^2	1	$N_H^{0.5}$	N_H^2	1
3	N_H^2/Q	Q	$(N_H/Q)^{0.5}$	N_H^2	1

Scaling law according to Case 1 fulfils the replica dynamic structural response law. A scaled-down model is said to be in replica mode when it achieves exact similarity in every senses of dynamic, materials and geometrical properties as the prototype, except scaled in size only. Although such scaling is reported by the authors to have caused distortion for tall structures, such distortions were negligible for short, stiff and strong structures. The only drawback is that the ground acceleration input has to be scaled inversely with the geometry scaling factor. In other words, the acceleration induced by shake table has to be increased to compensate for the modal's mass reduction.

Case 2 allows relatively lower ground acceleration input demand, thus requiring less force from shake table as compared to Case 1. Nevertheless, to balance out the lower ground acceleration, heavier mass is required. This should be achieved through either lumping additional mass onto the structure or choosing a material that is denser, but keeping the constitutive property. The latter is almost impossible to be fulfilled and impractical for small models. Lumping of additional mass does not truly represent the replica model compared to Case 1, due to the distortion of centre of gravity and stress distribution is changed. As a result, Case 3 is a compromise between the first two cases. Lumping of mass is still required, but lesser than Case 2 while lowering the demand of ground acceleration input than Case 1.

By examining each case closely, Case 1 scale factors were adopted in this study, since it represents the closest replica model to prototype structure. Both Case 2 and 3 were singled out because they were unable to produce the total mass of test model to be within the shake table's limitation weight capacity of 1,000 kg (1 ton). Listed in Table 3.3 are the scale factors for each dimensional element in scaling down the prototype structure in this study to construct the shake table test specimen. Due to the main concern was the limitation of mass, a geometrical scale factor NH of 3 was chosen to keep the total mass below 1000 kg. Thus, a scaling factor of 3 was selected to allow for possible downsizing of the prototype structure to testable size and weight by the shake table. The designed precast wall panels and column members for test specimens were as thin as only 50 mm in thickness. Such thin requirement in thickness of concrete element posed very high risk of honeycomb formation during concrete casting due to the presence of coarse aggregates. To compensate such unpleasant incident, the total specimen numbers were increased. Structural detailing of the one-third specimen is listed in Table 3.4.

Table 3.3. Scaling factors used in producing the shake table test specimen in this study

Parameter	Scaling factor
Mass	27
Acceleration	1/3
Time	3
Force	9
Material	1
Width (or Length)	3
Area	9

Table 3.4. Structural details of the 1:3 shake table testing specimen

Element	Section (mm)	Longitudinal Rebar (mm)	Shear Link (mm)
Wall (upper storey)	50 (t) x 800 (w) x 575 (h)	2 layers of BRC A7 (mild steel)	Not provided
Column (upper storey)	100 (t) x 100 (w) x 600 (h)	4R6	R4-70 c/c
Beam (upper storey)	50 (t) x 150 (d) x 1000 (l)	2R6 (T) 2R6 (B)	R4-60 c/c
Wall (lower storey)	50 (t) x 800 (w) x 875 (h)	2 layers of BRC A7 (mild steel)	Not provided
Column (lower storey)	100 (t) x 100 (w) x 600 (h)	4R6	R4-70 c/c
Beam (lower storey)	50 (t) x 100 (d) x 1000 (l)	2R6 (T) 2R6 (B)	R4-60 c/c
Concrete	Compressive strength = 30 N/mm^2		

4. SHAKE TABLE TEST

Laboratory setup can be found in Figure 4.1. Channel 1 served as the reference acceleration acquisition. Connecting this channel to the shake table itself was the Dytran accelerometer model 3165A, which had very high sensitivity of 1000 mV/G. The accelerometers used for channel 2 and 3 were both Dytran model 3100M14, having sensitivity of 100 mV/G. Meanwhile, channel 4 was the Kistler K-Shear accelerometer.

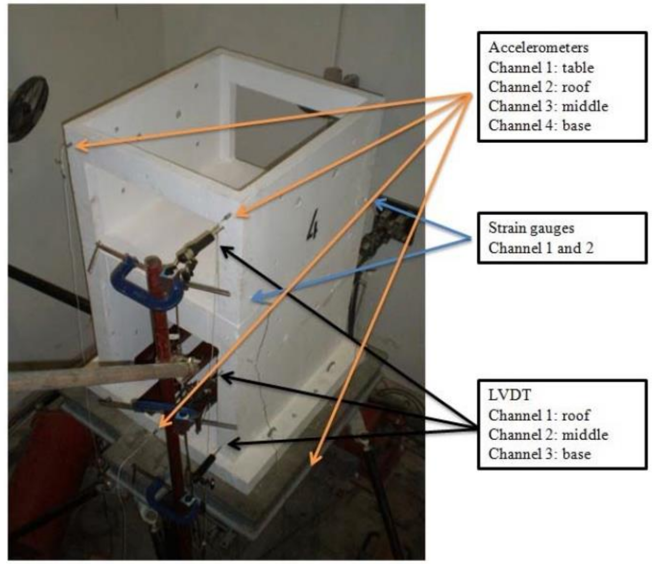

Figure 4.1. Installed sensors for data acquisition of the shake table testing of HCPS

Seven time histories were obtained from the database of Pacific Earthquake Engineering Research (PEER) as listed in Table 4.1. In addition to the seven real earthquake records, synthetic time history for Malaysia was added to the test. These time histories were selected to represent a broad category of soil condition, earthquake magnitude, and distance from epicentre to recoding station. Last but not least, the types of fault mechanism causing the earthquakes were also one of the determining factors.

Table 4.1. Detail of selected time histories for shake table test

Earthquake	Year	Station	Scaled PGA (g)	Fault type	Soil Type	Distance (km)
El Centro	1940	117 El Centro #9	0.96	SS	C	6.1
Tabas	1978	71 Ferdows	0.114	RV	C	91
Irpinia	1980	Bagnoli Irpino	0.606	N	A	8.2
Kobe	1995	0 Kakogawa	1.035	SS	C	22.5
New Zealand	1987	Matahina Dam	0.165	N	B	16.1
Taiwan SMART1	1983	28 SMART1 M01	0.117	RV	C	27.4
Duzce	1999	Ambarli	0.075	SS	D	189
Malaysia Artificial	-	-	0.606	-	A	400

Note: *SS* = strike slip fault; N = normal fault; RV = reverse fault

5. RESULTS AND DISCUSSION

The experimental results observed were roof acceleration and displacement responses of the 1:3 HCPS model through installed accelerometer and LVDT. Results of the strain gauges were excluded from this paper as the test specimen of HCPS remained within its elastic response in all the shake table tests which caused no significant reading recorded from the strain gauges installed.

Table 5.1 shows the summarized peak ground acceleration observed at the base of HCPS and the peak roof acceleration response. It was observed that amplification of ground acceleration occurred in all excitations except for Tabas earthquake in which the roof acceleration was 17 % lesser than peak ground acceleration. In all remaining seven time histories, ground amplification up to 2.4 times was observed in El Centro ground motion. The amplification of acceleration occurred because of the vibrating characteristic of the structure (HCPS) due to angular frequency.

Table 5.1. Laboratory observed acceleration response at base and roof of HCPS

Ground motion	Base acceleration	Roof acceleration	Amplification
	(g)	(g)	-
El Centro	0.67	1.61	2.40
Irpinia	0.56	0.65	1.16
Malaysia	0.51	0.60	1.18
Kobe	0.64	0.72	1.12
Tabas	0.43	0.36	0.83
Duzce	0.71	0.77	1.09
New Zealand	0.24	0.32	1.34
Taiwan SMART1	0.24	0.31	1.29

Table 5.2 shows the drift ratio observed from the shake table study. Interstory drift values of 0.01 % (minimum) and 0.16 % (maximum) were noted in Duzce and Kobe earthquake respectively. The roof drift was insignificant as the structure is considerably very rigid in its lateral direction.

Table 5.2. Laboratory observed displacement response at base and roof of HCPS

Ground motion	Base displacement	Roof displacement	Drift
	(mm)	(mm)	(%)
El Centro	25.30	26.68	0.08
Irpinia	4.33	5.18	0.05
Malaysia	14.85	17.28	0.15
Kobe	14.88	17.57	0.16
Tabas	9.75	8.80	0.06
Duzce	10.36	10.52	0.01
New Zealand	3.60	5.18	0.09
Taiwan SMART1	3.65	4.44	0.05

Comparisons between the roof acceleration response and drift between the specimen and FE models of the prototype structure can be found in Figures 5.1 and 5.2 respectively.

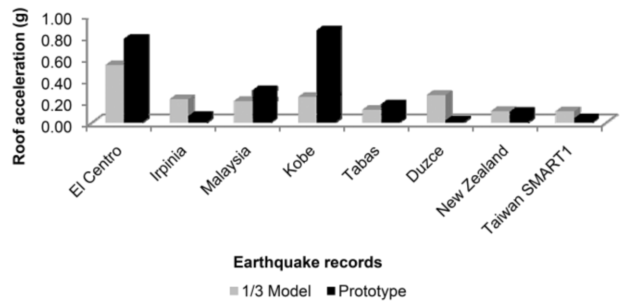

Figure 5.1. Comparison between specimen and prototype in terms of roof acceleration response

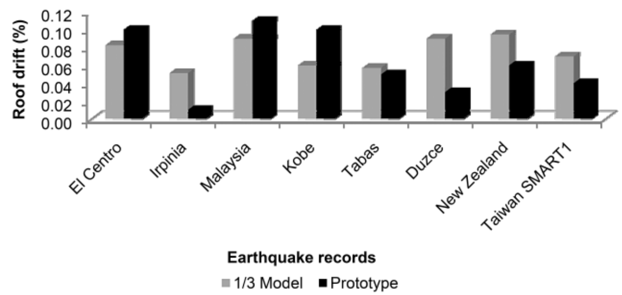

Figure 5.2. Comparison between specimen and prototype in terms of roof drift response

In general, the structural performance between both specimen and prototype structures were found to be in good agreement with each other in most of the earthquake excitations. Discrepancy was observed in Kobe earthquake records because the large acceleration force exceeding 1.0 g that was required in the shake table test was unable to be met by the shake table due to the relatively heavier weight of the HCPS specimen. Performance of the shake table degraded as the payload increased. Hence, the comparisons between the results were biased since the benchmark was obviously different.

6. CONCLUSION

In this study, shake table test of 1:3 scaled HCPS specimen was successfully designed and tested using seven real earthquake records and one synthetic time history of Malaysia. It was revealed in this study that despite showing some discrepancy in very large ground acceleration that fell beyond the capacity of the shake table, the scaled-down specimen was able to demonstrate rather reliable performance compared to the FE models of the full size prototype.

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