CTBUH Review

The Professional Journal of the Council on Tall Buildings and Urban Habitat

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CTBUH Review is the Professional Journal of the Council on Tall Buildings and Urban Habitat. It includes refereed papers submitted by researchers, scholars, suppliers, and practicing professionals who are engaged in the planning, design, engineering, construction, and operation of tall buildings and the urban environment throughout the world. *CTBUH Review* is published bi-annually with the generous support of the journal sponsors.

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From the Editors:

CTBUH Review

Welcome to this inaugural issue of the *CTBUH Review* – an electronic journal dedicated to providing architects, engineers, planners, developers, contractors, manufacturers, social scientists, and other professionals throughout the world involved in the planning, design, construction, and operation of tall buildings with state-of-the-art research and information.

CTBUH Review will be published biannually by the Council on Tall Buildings and Urban Habitat. The scope of the journal is defined by the eight principal groups of the Council.

There is a need for a professional journal which addresses the specialized requirements of tall buildings and their impact on the urban environment.

Tall building engineering and technology was developed in the United States and it made possible the great high-rise metropolises of the modern city.

Today, new technologies for tall buildings are being developed by engineers and architects and applied to tall buildings throughout the world. With burgeoning populations in developing countries and reallocations of economic and environmental resources, major international cities have adopted high-rise architecture as one of the preeminent building types. Publication of this new journal through the electronic medium at the cross-roads of anew century and a new millennium thus makes it truly exciting.

Although advancements in engineering and technology have historically redefined the design and construction of tall buildings, the impact of tall buildings on the local ecology and urban habitat of metropolitan centers is already reshaping the way tall building architecture is approached. The complexity of tall building systems, client and user groups, impact on local and regional infrastructures, initial versus life-cycle costs, aesthetic considerations, and so on, require an interdisciplinary design approach which encourages collaboration of many diverse design professionals who can bring their own expertise into the programming, design, and construction process.

The purpose of this journal, therefore, is to provide an opportunity for dissemination of critical research and information in the areas of planning, design, and construction of tall buildings and urban habitat to an international audience of engineering, construction, architectural, and planning and other professionals. *CTBUH Review* is committed to professional and academic excellence. All submissions of original research will be thoroughly reviewed for content prior to publication. All academicians and professionals interested in tall buildings and the related urban environment are invited to submit original research material and practical case studies.

Authors are requested to join the group of experts by contributing to this journal. Manuscript guidelines are available either through the Council headquarters or may be requested directly from the Editors. Prospective authors are requested to follow these guidelines as strictly as possible.

Papers will initially be screened by the Editor and Co-editor. Each paper provisionally accepted will be sent to two reviewers who will be provided with manuscript review guidelines and a review form. The reviewers may be from the Editorial Advisory Board or may be outside reviewers from the area of the author's specialization. Authors will be notified of acceptance and of any major changes that must be made to the text prior to publication.

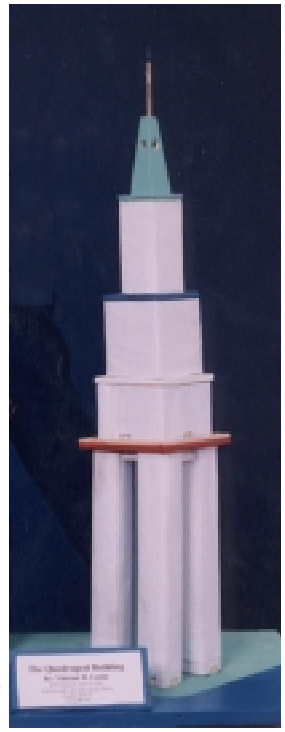
Discussions on papers are most welcome. Also, opinions on any relevant issues may be expressed through the "Letters to the Editor" column. We would very much like to hear from you about your views on the journal and on any other topics pertaining to tall buildings and the urban habitat.

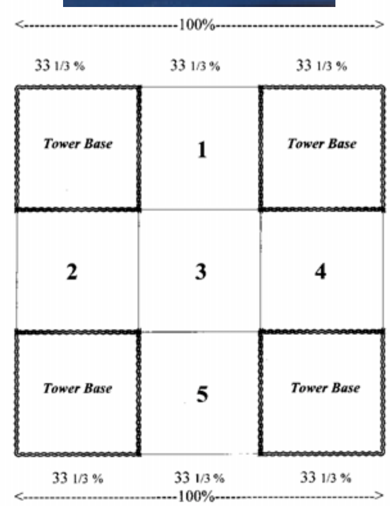
Discussions and letters may be addressed to the Editor at inctbuh@lehigh.edu.

Mir M. Ali, Editor
Paul J. Armstrong, Co-editor

Letter To The Editor:

A New Structural Design Concept: The Quadruped Building





Quadruped Building Layout Diagram #1

To the Editor:

I am a 1952 Civil Engineering graduate of Rensselaer Polytechnic Institute and since retiring have become a member of the Council on Tall Buildings & Urban Habitat. However, I am not an architect, and am in search of an architectural or structural engineering firm interested in collaborating on the design and erection of The Quadruped Building.

I have enclosed a photograph of a model of my concept of a Tall Building which if constructed will be the tallest building in the world, twice as high as the Empire State Building.

Theoretically, steel-frames and high speed elevators permit the rise of skyscrapers to unfathomable heights. In reality, of course, engineering height and economic height are intertwined.

The proposed Quadruped Building is 2800 feet in structural height, yet the design concept decreases the cost of construction since the space between the four Quadrupeds or Towers is not included in the construction costs.

In the diagram below, the layout of the four towers is shown on the building plot divided into nine squares, with the tower bases occupying the four corners. The building design calls for the remaining five squares to be without interior building construction. Thus, the total square footage of construction required for a solid building base would in effect be reduced by five-ninths. When calculated to include labor and materials required to erect the exterior walls of the four base towers, and interior court yards, the actual cost savings would be slightly less.

Essentially, the Quadruped Building concept based on the premise that in order to soar above all the oter high buildings, one must first erect four 1400 foot Towers upon which additional sections shall rest.

The Towers, set on bedrock and braced, are approximately 60 feet apart, on top of which is a 32,400 square foot platform capable of supporting three additional sections, consisting of 300 feet 400 feet and 500 feet, as well as a 200 foot light tower and the video antenna.

As illustrated on the photograph the first 300-foot building provides space for a 360 degree observation platform. The enclosed sections lend themselves to commercial uses such as restaurants, banquet halls, theaters, retail outlets, fine art facilities, and arenas.

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CTBUH: A Historical Sketch

Lynn S. Beedle

It all started with the IABSE. It is not often that one can remember the precise start of something...right back to the moment he had the idea. But in the case of the Council on Tall Buildings and Urban Habitat, I can: The afternoon of Friday, September 13th 1968, at the 8th Congress of the IABSE in New York. Prof. H. Beer of Austria was summarizing the theme. "Tall Steel Buildings," and I was struck by the significant tall building research he was describing. This research was not being coordinated or evaluated in a form useful to the designer. It spoke of the need for an international effort to bring information together.

Acting upon that idea, I wrote to Prof. Beer and in due course he responded that international exchange indeed should be started. By that time, it was February, 1969 and at a meeting of the US Group IABSE in New Orleans, then chairman Elmer Timby, asked us to suggest a topic that we could "gather around" as a basis for more frequent exchanges with our professional colleagues overseas. Here was the opportunity to implant the idea: the preparation and updating of a Monograph that would provide a focus for continuing exchange. It would be a joint activity between the IABSE and ASCE – hence its original name, "Joint Committee on Tall Buildings."

Jewell Garrlets made the presentation to IABSE at its following meeting in Britain in September of 1969. Approval by the American Society of Civil Engineers came shortly afterwards, National Science Foundation funding was approved, and we were on our way. The Headquarters was established at Lehigh University, Bethlehem, PA, USA.

The need for the Joint Committee was more than just the desire to get together. It stemmed from things like the exploding urban population, creating an increased demand for tall buildings; the need for economy in construction; the frequent neglect of human factors at the expense of livability and the quality of life; the need for new research required in the field, and the necessity of establishing priorities for such research.

The timing was right, too. There were very few high-rise buildings built in the 1950s. But by the 1970s, tall building construction increased substantially.

As a result of the increased emphasis on planning and environmental criteria in 1973, the American Institute of Architects, the American Planning Association, the International Federation for Housing and Planning, and the International Union of Architects were invited to join the forming organizations as equal participants with IABSE and ACSE. Since then, the American Society of Interior Designers, the Japan Structural Consultants Association and the Urban Land Institute also have become sponsoring societies. Then, in 1976, the "Joint Committee" changed its name and became known as the Council on Tall Buildings and Urban Habitat.

In 1979 it was admitted as a Category B non-governmental organization of UNESCO.

One of the well-remembered Steering Group meetings was one of the first, at which Les Robertson and the late Fazlur Khan were debating the question of "What is a tall building?" After all, if we were going to do a Monograph on tall buildings, one needed a definition. The final decision: A tall building is not defined by its height or number of stories. Rather, the important criterion is whether or not the design is influenced by some aspect of "tallness." It is a building whose height creates different conditions than those that exist in common buildings of a certain region or period.

The Steering Group next organized the 1971 conference in Bled, Yugoslavia, to bring together specialists from all over the world to decide what this Monograph would be all about. Delegates reviewed the abstracts of papers that later would be presented at the First International Conference (they are now called World Congresses) held at Lehigh University. More than 700 people attended this later five-day event from August 21 through 26, 1972. Adhering to a strictly enforced 7-minute time limit, 261"reporters" and over 200 discussers participated, coming from 30 countries. Twenty-seven preprint volumes were available to the participants, followed by a five-volume set of Proceedings. It is still known around the world by council members as the "Lehigh Conference."

That conference was followed by an intensive series of follow-up conferences – 20 being held in the 1972-1975 period. Their essential function was to disseminate the information coming out of the Lehigh Conference and to collect material for the Monographs.

Throughout its over 20-year history, the Council has continued to strive toward the dissemination of information and the stimulation of research on tall buildings throughout the world. It continues to have a major concern with the role of tall buildings in the urban environment and their impact. It is not an advocate for tall buildings per se; but in those situations in which they are viable, it seeks to encourage the use of the latest knowledge in the implementation. In addition to sponsoring conferences on a regional and international basis, the Council has continued the work of the original Monograph, publishing update volumes, a regular newsletter, and other reports and support documents. Our Fifth World Congress is planned for Amsterdam, scheduled for May, 1995. A complete new Monograph series is underway, aimed at documenting the latest research and development in the field of tall buildings – and their impact on the urban environment and the people who dwell therein.

Much has been accomplished by the council, but most important, perhaps are the relationships that has been established with colleagues from a wide range of professions-and from all over the world. We thank Prof. Beer for lighting the spark on September 13, 1968. And we thank Elmer Timby for asking the right question.

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The Ecological (or Green) Approach to Design

Ken Yeang
T. R. Hamzah & Yeang Sdn. Bhd.

Abstract

Described here are some of the criteria for ecological design. However, the designer is advised that ecological design is not just meeting these criteria but to be as comprehensive as possible in ensuring that design has the least descriptive disruptive impacts (or the most beneficial impacts) on the ecosystems and non-renewable resources in the biosphere.

Introduction

A guide to the connectivity in ecological systems is in the model below (Yeang, 1997).

Ecological design involves the holistic consideration by design, of the careful use of energy and materials in a designed-system, and the endeavourendeavor by design to reduce the impacts of this use on (and its integration with) the natural environment, over the life-cycle of the designed-system from source-to-sink (Yeang, 1997).

We can structure these considerations in a framework of set of interactions (vis-a-vis impacts) between the built environment and the ecological environment. These interactions are analogous to the concept of an open system. Based on the above features, interactions can be classified into four general sets:

o The external interdependencies of the designed system (its external or environmentall relations)
o The internal interdependencies of the designed system (its internal relations)
o The external-to-internal exchanges of energy and matter (its inputs)
o The internal-to-external exchanges of energy and matter (its outputs)

In an ecological approach to design, we must simultaneously consider all four of these aspects as well as their inter-relationships with each other.

These interactions can be further structured into a symbolic form as follows:

Given a designed system and its environment, let suffix 1 denote the system under the consideration and suffix 2, the environment around that system. Further, let letter L be the interdependent connections within the framework.
It follows that four types of interactions can be identified in the analysis: L11, L12, L21 and L22.
This can be further represented in the form of a partitioned matrix (LP) as follows:

$$(LP) = \frac{L11 \quad L12}{L21 \quad L22}$$

L11 refers to the processes and activities that take place within the system or the area of internal interdependencies.

L12 refers to the processes and activities that take place in the environment of the system, or the external interdependencies.

L21 refers to the exchanges of the system with its environment, or the transactional interdependencies of the system / environment.

L22 refers to the exchanges of the environment with the system, or the transactional interdependencies of the environment / system.

The above also provides the broad theoretical basis for ecological design. The holistic nature of the above model could be found in the traditional geomancy Feng Shui.

Key Criteria

Key criteria for ecological design are:

Issues	Design Target
1. Assess need for building? • Place priority on users over hardware. • Assess levels of provision of internal environmental systems. [or the Internal Interdependencies]	Evaluate Design Brief and basis for the project and user requirements on ecological sensibility before embarking on design. • passive mode • level of servicing
2. Assess where it is to be built? • Ecological Value of Site (by building configuration) [or the External Interdependencies]	Site planning (i.e. location of buildings, cut-and-fills, roads, paved areas, etc.) should be based on 'ecological land use method'. Buildings are to be located on those parts of the site with minimum disruptions and impacts on the locality's ecosystem. Enhance the ecological value of the site with locally characteristic flora and fauna. • Assess ecosystem hierarchy of project site. • Ecosystems Hierarchy & Design Strategy • Ecological Land U~~u~~se Planning / Sieve Map Technique [for ecologically-mature to monoculture project site] • Evaluate Biodiversity Index before and after construction: • Species Diversity in Relation to Latitude
• Assess Local Wind Effects (by building configuration).	Design to limit frequency of exceeding wind levels of 4 or more on the Beaufort scale to 20% or less (i.e. 5.5 to 7.9 m/s) to reduce impacts on pedestrians and on surrounding sites.

Beufort No.	Windspeed (m / sa)	Description	Land Condition	Comfort
0	0 - 0.5	Calm	Smoke rises vertically	No noticeable wind
1	0.5 - 1.5	Light air	Smoke drifts	
2	1.6 - 3.3	Light breeze	Leaves rustle	Wind felt on face
3	3.4 - 5.4	Gentle breeze	Wind extends	Hair disturbed, clothing flaps
4	5.5 - 7.9	Moderate breeze	Small branches in motion, raises dust & loose paper	Hair disarranged
5	8.0 - 10.7	Fresh breeze	Small trees in leaf begin to sway	Force of wind felt on body
6	10.8 - 13.8	Strong breeze	Whistling in telegraph wires, large branches in motion	Umbrellas used with difficulty
7	13.9 - 17.1	Near gale	Whole trees in motion	Inconvenience in walking
8	17.2 - 20.7	Gale	Twigs broken from trees	Progress impeded Balance difficult in gusts
9	20.8 - 24.4	Strong gale	Slight structural damage (chimney pots and slates)	People blown over in gusts
10	24.4 - 28.5	Storm	Seldom experienced inland. Trees up-rooted, considerable structural damage	

Windspeed & Conditions (Measured 10m above sea or ground level)

• Check overshadowing of other buildings and land (by building configuration)	Locate buildings on the site to avoid substantial overshadowing of neighbouring buildings and land. Building configuration can be based on the site's 'solar envelope' so as not to overshadow the solar production potential of neighbouring sites. This affects solar-energy potential of adjoining sites (and their conditions in winter months for temperate climate sites).
• Check outdoor Noise (by building configuration)	Design to ensure that the noise rating levels outside the nearest exposed residential buildings are not less than 5dB below the background level during any period of the day or evening (0700-2300 hours) and does not exceed the background level during any period of the night (2300-0700 hours). The impact of this is mainly on users.
• Other aspects (e.g. emissions, transportation, etc.)	Check ground level vegetations for leaf discolouration, etc.

3. Assess what is to be built? • Embodied energy and CO_2 in building materials (from building production) [or the External-to-Internal Interdependencies]	Consider the use of energy embodied values in materials in construction (and maintenance) taking account of the full life cycle of the material in question and their recovery (e.g. reuse / recycling) potential. Consider the incorporation of locally-sourced materials for major building components. Overall embodied energy and embodied CO_2 for various building types are to be within the following standards:

Building Type	Embodied Energy Delivered GJ/m^2	Embodied Energy Primary GJ/m^2	Embodied CO_2 kg CO_2/m^2
Office	5-10	10-18	500-1000
House	4.5-8	9-13	800-1200
Flat	5-10	10-18	500-1000
Industrial	4-7	7-12	400-700
Road	1-5	2-10	130-650

• Assess environmental impacts of production of building (from building production)	Check the environmental impacts of the flow of source of materials and energy in the production of the building and their impacts.
• Assess natural resources consumption and Recycled Materials (from building production)	Material specification to consider depletion of natural resources. (e.g. Timber/timber products from sustainable sources, etc.). Demolition materials to be re-used (if appropriate)

• Assess hazardous materials (from building production)	Avoid the specification of materials known to be hazardous wherever possible and / or where no economic alternative specification is available (e.g. wood preservatives).
• Avoid over specification (from building production)	Monitor performance.
• Assess building construction impacts (from building production)	Building construction work should not cause ecological disruptions to the ecosystem within site and to those in surrounding sites. The long term success of a 'sustainable' design also requires particular attention to the construction, commissioning and the monitoring and control of the building use. The environmental performance requirements of the Contractor will be written into their Contract to include: • Developing and implementing a project environmental plan. • Minimising waste. • Minimising re-work. • Efficient use of energy and other resources. • Preventing pollution. • Using recycled and recyclable materials or components, where possible. • Minimising the need for transport, including importation / exportation of materials. • Proper disposal of unavoidable waste, including full compliance with relevant regulations, and clearing the site on completion.
• Assess external landscape design (from building production)	Micro climate amelioration can be achieved by site planning and landscape design. The following factors and techniques will be explored in further design developments. Inter-related factors which form local micro climate. • solar radiation • temperature • relative humidity • evaporation • wind • precipitation Four main elements affect human comfort - solar radiation, air temperature, air movement or wind, and humidity or precipitation. When the combination of these do not place undue stress on the human, conditions reach the human 'comfort zone. In the UK, this zone lies between 14-21˘C. The close the outdoor climate can be kept to this range the less energy is consumed to produce a comfortable indoor climate. The form of the landscape can have a beneficial physical effect on the energy consumption of buildings, will therefore reduce costs and reduce the micro climate.

	The landscape design should aim at ameliorating the micro climate of spaces around buildings to provide more comfortable conditions for people to use these spaces.
4. Assess impacts of building operations?	• Optimise on passive energy systems through use of ambient energy by: • Builtform configuration • Builtform orientation • Facade Design • Solar Control Devices • Builtform envelope colour • Vertical Landscaping • Wind and Natural Ventilation • To design a simple control strategy that all building users understand and can contribute to controlling. Delivered energy use in a typical air-conditioned office building
• Optimise use of non-renewable sources of energy e.g. photovoltaics	
• Reduce Carbon Dioxide Production (due to energy consumption) (from building operations) • Acid Rain (from building operations)	Control CO_2 production. < 50kg/m^2 per year< 200mg/kwh delivered energy for reduced NO_x emitting boilers.
• Ozone Depletion (due to CFCs, HCFCs, and Halons) (from building operations)	• Zero ozone depleting refrigerants. No halons. • Zero ozone depleting insulation for fabric and services.
• Design for storage of Recyclable Materials (from building operations)	• Provide adequate space for separate storage of wastes for reuse / recycling collection (as appropriate).
• Assess avoidance of Legionnaires' Disease (arising from Wet Cooling Towers) (from building operations)	• Avoid wet cooling-towers or design to CIBSE TM13 • Design hot water systems to meet specifications of CIBSE TM13.
• Check Enclosure Insulation Values (from building operations)	• The insulation U-values for walls and roofs should be less than 0.35W / sq.m.. The minimum air-leakage standard @ 5 cu.m. / hr / sq.m. of facade at 50PA.
• Assess Building Life-cycle consequences (from building operations)	• Assumed building life-cycle is 50 years as the basis for all calculations. Life-cycle energy costs of a built system. We should note that the system operational costs over its life span far exceed the costs in its other stages.

• Place priority on facade design over content.	• The facade design has a crucial role in energy performance.
• Optimise passive systems for wind and Natural Ventilation, Passive smoking and Humidity (from building operations)	• Design majority of space as naturally ventilated. • Prohibit smoking in the building. • Avoid need for humidification plant. • Careful design of humidification system if required in non naturally ventilated areas (i.e. steam-based system). • Indoor air quality should be at c. 25 litr. / person (i.e. full fresh-air with no recirculation) • Make use of natural ventilation devices (e.g. displacement systems, wing-wing walls, etc.).
• Optimise passive natural and artificial lighting systems (from building operations)	To provide good levels of visual comfort by lighting in the office: • maximise area of daylight working space to aim for at least 80% of all the area used for office work to meet the daylighting criteria set out in British Standard BS 8206:Part 2 (Code of Practice for Daylighting). • high frequency ballasts should be fitted to all lamps with fluctuating output (e.g. fluorescent). • use light-shelves, special glass, etc.
• Check thermal comfort and overheating (from building operations)	• Minimise the risk of discomfort due to overheating by incorporating passive design features to complement the proposed natural ventilation and night cooling strategy, perform satisfactorily by calculation consistent with CIBSE Guide Volume A (Section A5).
• Control indoor noise (from building operations)	Achieve comfortable acoustic conditions in the offices and meeting areas: • private offices and small meeting rooms - $40dBL_{arql}T$. • large offices - $45dBL_{arql}T$. in accordance with BS 8233:1987 sound insulation and noise reduction for buildings.
• Examine life-cycle energy consumption (e.g. trade-off energy efficiency with embodied energy cost) (from building operations) [Internal Interdependencies]	• Minimise total operational energy consumption, which is typically the highest component of building energy use. Operational, the energy consumption for the building should be c. 30% to 15% less than the conventional office building (nb. the office building is taken as the maximum consumption standard for the high-rise tower). If the modern air-conditioned office tower's energy consumption per annum is c. 400 kWhr / sq.m. / annum, then acceptable standards should be c. 65 to 125 kWhr / sq.m. / annum. This might be achieved through composite means (e.g. part natural ventilation, solar shading, flue walls, etc.), by efficient systems and application of renewable energy sources (e.g. photovoltaics, solar energy, etc.).
• Assess water pollution (from building operations)	• Reduce Site run off to provide at-source management of pollution from surface water run-off. • Improve absorption and return to ground water.

• Assess transport energy (from building operations)	• Minimising car parking on site. • Policies to encourage public transport and discourage car use. • Consider staff transport schemes including incentives
• Assess Indoor Air Quality (IAQ) (from building operations)	• Particulate filtration. • Assessment of air quality / CO_2, Furnishings / maintenance • Provide natural ventilation by openable windows capable of maintaining CIBSE recommended air change rates. • Avoid air circulation. • Use indoor plants to absorb VOC's.
• Reduce Sick Building Syndrome (from building operations)	• Occupant surveys / monitoring • Review performance of building-in-use annually.
• Design for Internal Water Conservation (from building operations)	• Design to aim to reduce water requirement from mains to drinking water only. • Water recycling proposals, rainwater use, waste water treatment proposals / use of sewage sludge? • Water efficiency management strategy, water leakage. • Irrigation systems to landscaping areas should use grey water. The rainwater should be collected and recycled (e.g. for toilet flushing).
• Reduce or recover waste material and heat (from building operations)	• Design to recover waste materials and heat from building operations. • Allow for transit storage of materials for recycling. • Design to avoid waste.
• Check Internal Fit-Outs (from building operations)	• Base building to accomodate relocatable meeting rooms and business centres. • Furniture layouts to allow flexible team arrangements with minimal alterations. • Check use of hazardous materials.
• Design for disposal of building demolition materials and excavated material (from building disposal)	• Minimise amount of material taken off site (aim for none). • Reuse and Recovery • Reuse and Recovery where
5. Assess recovery of all materials and equipment after useful life of building? [Internal to External Interdependencies]	1. recovery within production processes 2. recovery of construction residuals for construction purposes 3. recovery patterns in consumption 4. recovery of consumption materials into production processes 5. recovery of construction materials into production processes 6. recovery or redirection of materials elsewhere A cyclical pattern of use

Conclusion

The above is prepared as a simple guide for ecological design using some of the prevalent standards. The designer is cautioned that this list is not totally comprehensive and that standards change as technology and theory of ecological design develops.

References

Yeang, Ken, 1997. "The Skyscraper Bioclimatically Considered," *Architectural Review,* v202, August 1997, p. 88.; also published in Transition, no. 56, pp. 72–75.

The South of Market Area (SOMA) San Francisco: From Dross to Gloss

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Abstract

A large area of downtown San Francisco adjacent to the Financial District, usually called SOMA (the South of Market Area) is undergoing a transformation that appears to be accelerating every week. For nearly one hundred years the area stood in desolation whose only purpose seemed to be as a place for cars to enter and exit freeways taking commuters in and out of the city. This rapid ascent in real estate values and building construction is being fueled by the high-tech sector of Silicon Valley. More and more companies are interested in having a second office in the city of San Francisco, and many start-up companies are desirous of having their workplace conveniently located within the city. And as it usually happens, when a particular sector of the economy becomes strong in one area, the rate of expansion can happen very quickly. Computer companies, software companies, and internet start-ups attract more of the same. In addition, the youthful flavor of the neighborhood is continuing to attract more of the young professional crowd for both work and living. Owners, developers and construction firms like what they see and are hopeful this trend continues.

The objectives of this paper are to examine the urban redevelopment currently taking place in the South of Market Area (SOMA) in the city of San Francisco and to contrast the Gold Rush years in San Francisco (1849-1854) with the current changes occurring in South of Market Area. The paper concludes with an architectural case study of Avalon Towers, a new development that has revitalized the South of Market Area.

Introduction

The South of Market area has worn many faces economically and culturally since the beginning of the Gold Rush in 1849. It has seen its residents gyrate from the lowest level of society to the highest in a period of less than five years, and then slide dramatically downward in a period of one year. In the period between 1855 and 1870 the neighborhood called Rincon Hill was home to the wealthiest sector of the city, with some houses having as many as forty rooms. The year 1869 was the beginning of the downward descent when a deep cut was introduced by a misguided politician, to shorten a route to the South, that created such decay that the entire area was slowly abandoned. With the assistance of the newly created Cable Car, the rich slowly took over Nob Hill as a preferred residential area. Rincon Hill was sliced away bit by bit until not a trace existed of it. The twentieth century saw a rotting decay of the entire South of Market Area that only began to end in the 1990s.

Fig. 1. Avalon Towers – Computerized Model

The Re-Development Process. The re-birth of the area began approximately five years ago and the transformation has followed the normal pattern of speed in the typically San Francisco spiral mode. The new technology age of development that gave birth to near-by Silicon Valley has fueled an economic expansion that is unique to the San Francisco Bay area. The turn around could be described as "rubble" turned to "riches."

Avalon Towers: Catalyst for Change

The catalyst in this high-impact change is the new Avalon Towers residential high-rise complex at 388 Beale Street (Fig. 1). This is the first residential rental high-rise building to be completed in the city in more than twenty years. Designed by the architectural firm Theodore Brown & Partners, the 225-unit luxury building was completely leased within ten weeks of completion. It has been called the Internet.com building since a large percentage of the tenants are employed in the computer field. A New York Times article (August 29, 1999, p. 44) profiled The Towers in a half page article. It stated that 85% of the residents earn more than $100,000 per year, 75% are single or divorced, 60% are under 40 years of age, and two-thirds are new to the city of San Francisco.

When Theodore Brown was asked to design the Avalon Towers in 1990, no one could have envisioned the clientele that would be resident. The word Internet was an obscure scientific endeavor until the innovation of the World Wide Web became a reality in 1993.

Since San Francisco is located very near to Silicon Valley, a region reported to produce 64 new millionaires every twenty-four hours, the South of Market Area is poised for an expansion of would-be gold seekers. Gold is in the blood running through these veins! In a job market that is shifting from seniority-based pay to personal-based pay, the new young employee is driven to put in more hours at work each week. Companies are now closely evaluating if these hours are productive. Recent trends indicate companies now offer stock options and bonuses that enable them to lure and keep talent without offering large base salaries. The 40 hour work week has about disappeared as most American workers now log in more than 260 hours of work a year than they did a decade ago.

Therefore, living close to work or close to a freeway access such as the South of Market Area provides is now seen as a positive for the new employee. The trend toward suburban living is not as attractive to the employee who logs in up to 80 hours of work each week.

The South of Market visual landscape is becoming dotted with restaurants, as the new employee working in the area has no time to leave to search for a place to eat lunch or dinner. The casual lifestyle of the computer industry in Silicon Valley has been brought into the center of San Francisco. The pin-stripe business suits usually remain North of Market in the Financial area while the South of Market restaurants follow the Silicon Valley casual style. The social and business atmosphere is fast-paced, intense, highly competitive, youth-oriented, and ephemeral as both jobs and relationships are oriented toward change.

Scaffolding and cranes now define the area visually. New buildings are sprouting up quickly. A few of the major ones are the Pacific Bell Park (home of the San Francisco Giants), 88 King Street (a high-rise residential project), the new Gap Corporate Headquarters on the Embarcadero, the Courtyard Marriott, the Brandon City homes, 246 Second Street (a residential high-rise), Pankow Residential (a high-rise), the Telecom Center, the Townsend Building and a dozen or more live / work lofts modeled after the So-Ho concept in New York City. In addition, many warehouses and last century factories that have stood with gaping windows and crumbling roofs are suddenly becoming conversion projects for either retail or office use. A number of streets have had to be closed because of extensive construction projects. Sports clubs, bicycle and sports shops, chiropractors with sports medicine specialties and anything to do with competitive sports define the area. The new employee is as competitive in his or her personal life as in business. While the Financial area may close at 10:00 p.m. most evenings, the South of Market Area is more like a European city that remains alive into the wee hours of the morning.

Historical Aspects

The word "millionaire" may have been coined in San Francisco in the 1850s during the Gold Rush. Certainly the concept of instant wealth is not new to this area. It seems a second Gold Rush may be underway as more and more people globally arrive to seek out new opportunities. Nowhere in the United States is there a major city undergoing the transformation now visible in San Francisco.

The Gold Rush of the 1850s was the first truly global event of the world. Gold seekers came from every corner of the map. San Francisco became famous overnight! Psychologically, the Gold Rush changed every immigrant landing in San Francisco into becoming a new man in his own estimation. New arrivals were so convinced of their chance to become rich, that doctors and dentists became draymen, barbers or shoeblacks and lawyers and brokers became waiters and auctioneers. The Gold Rush mentality gripped the immigrant and no sacrifice was too great for this search for wealth. A similar trend has been occurring in the late 1990s as the Bay Area is attracting people from all over the country. These new workers (so-called twentieth century gold diggers) often must settle for inferior wages with the expectation of future riches. Working permits for foreigners are considered a coveted prize.

Rincon Hill. A large portion of the area being redeveloped in South of Market was once home to wealthy San Franciscans as they looked to find a desirable place to live. Rincon Hill rises 100 feet high and, in the mid-1800s, was laced with an abundance of springs, plenty of sunshine, was regarded by many as the best place to escape the fog and wind that characterizes other areas of the city. The streets were given English countryside names such as Hawthorne, Essex, Dover, and Laurel Place and were lined with wall-gardens and grassy areas. Living in this area were the lawyers, sea captains, bankers, senators, foundry owners, real estate investors, etc. There was no typical Rincon Hill estate as each house was unique; however, the Gothic style architecture was popular (Fig. 2). The shopping area near Rincon Hill was Second Street. Fancy ladies shopped at the milliners, jewelers, mantle makers, and ladies tailors, and of course, paid a visit to the hairdressers. However, all of this was short-lived. The area became developed between 1855 and 1860 and by 1869, it was declining rapidly. In the late 1860s a rumor circulated that a "cut" might be made through the heart of Rincon Hill. The Hill was considered to be a hindrance to commerce between the Pacific Mail docks and the mills, foundries, docks and wharves that existed below Market Street and the waterfront. It also blocked plans for future traffic to the proposed Central Pacific Railway terminal.

John Middleton, a real estate developer, was the promoter behind the scheme to achieve the Second Street cut. He outlined an access to be achieved by cutting Second Street to grade directly through Rincon Hill. There was some objection from some of the residents so Middleton obtained a Supreme Court order compelling compliance. Most of the city was more interested in expansion and commercial progress so no vehement opposition occurred.

Within a year, 500 workers had gutted out a chasm that divided the hill and extended to a depth of 75 feet (Fig. 3). Heavy rains in the year of 1869 added to this disaster and the wealthy moved quickly off the Hill (Muscatine, 1975). This moment defined the beginning of the decline of the entire South of Market Area.

South Park. An Englishman, George Gordon, a successful builder, became a bit homesick for his native London. In 1852 he assembled a series of building lots on the south side of the South of Market Area which was described as the only level spot of land free from sand in the city's limits at the time. Gordon's plan was to lay out ornamental lots using the plans of squares, ovals, or crescents based on urban plan types in London. He was a man of taste and, therefore, desired a controlled environment with no stores, warehouses, or saloons. The lots were to be used strictly for residential purposes. By 1854 Gordon had acquired twelve acres for the South Park project. A well-known English architect executed designs and an oval garden served as a private gated park for the residents of South Park (Shumate,1988). South Park still exists today although most of the houses are not original. With the new wealth in the area, it is possible that the day will come when the entire project will be restored to its original glory.

Foreign-Born Population. International residents are not new to San Francisco. In the first year that South Park was completed, 2,000 enthusiastic men from England, France, and Italy gathered in the area to celebrate the end of the Crimean War. The fact that this was such a major event speaks for the number of foreigners living in San Francisco at the time.

Fig. 2. A Home at Rincon Hill
(Courtesy: California Historical Society)

Fig. 3. The Second Street Cut
(Courtesy: California Historical Society)

Economic Factors. High prices with great volatility are nothing new to the San Francisco area. During the Gold Rush years, poorly constructed houses were renting for $800 per month; a dozen eggs were $12.00; a loaf of bread was fifty cents (in the rest of the country, six cents). The cost of doing laundry was so expensive that it was not uncommon for it to be shipped overseas for cleaning or to be merely thrown away. Luxuries were in great demand and no price seemed too exorbitant for the newly rich. There was an immediate influx of merchandise from all over the world in response to the high demand for goods. Everyone became an instant businessman. Ships found their way to San Francisco and goods were stacked on the beaches or on streets while warehouses and wharves were being constructed at lightning speed (Muscatine, 1975).

The Gold Rush occurred officially between 1849 and 1853 when it peaked. In 1854, the first of many subsequent depressions occurred. Some of the depressions were caused by a high influx of foreign labor willing to work for a small fraction of the normal wage scales.

Social Factors

Women were in short supply for the first several years of the Gold Rush. Only about eight per cent of the inhabitants were female in this period. Today male employees far outnumber women in the computer industry. A recent article in The International Herald Tribune (April 11, 2000, p.11), for example, points out that 36% more males than females live in the Silicon Valley town of Palo Alto. It also mentions that males involved in the computer industry are usually occupied in their own world of cyberspace and do not have much time for social activities.

Architectural Aspects of Avalon Towers

Avalon Towers (Figs. 4,5) is a development of AvalonBay Communities, Inc. The 20-story, twin-towered, 227-unit luxury apartment complex contains over 500,000 square feet. It is a state-of-the-art apartment building featuring a multi-purpose activity room, a spa and health club, parking for 230 cars, computer hook-ups for all units, and a landscaped plaza area for tenants' enjoyment.

Avalon Towers is the first high-rise, rental-only apartment building constructed in San Francisco in 20 years and designed to the 1997 seismic structural code. This project incorporated the Mayor's Office of Housing and the Planning Commission's policy for affordable housing by providing 23 affordable housing units on-site.

With easy access to the Waterfront Promenade along the Embarcadero on the southeastern waterfront and to downtown San Francisco, the location of Avalon Towers encourages journey-to-work trips by foot, bicycle or short local transit. The location also provides easy vehicular access to regional freeways.

The project exceeds the amount of open space required by the San Francisco Planning codes. It includes an access stair and ramp from Harrison Street to Beale Street via a pedestrian walkway (mid-block, open-air) as per the Rincon Hill Plan. This pedestrian street has the feel of a plaza / urban park with extensive landscaping, fountains, decorative paving and seating. The open space enhances the project site and contributes to making this area a desirable residential neighborhood.

The architects, Theodore Brown & Partners, Inc., have designed a visually interesting building structure using an exterior exposed concrete moment frame that soars, cantilevers, and steps skyward to a height of twenty stories. The stepped profile of the towers allow balconies and terraces for each unit. Most of the units are corner plans designed for panoramic views. The main building entrance is centered in a four-story curved, "Calcutta Gold" marble wall that connects the two towers. Penthouse units on the top three floors will have grand terraces, high ceilings, wood burning fireplaces, and granite countertops. Full-height glass fills in the void between the concrete frame and floor slabs. Dramatic triangular balconies cantilever from the structure. All of these features allow the residents to enjoy the views and fantastic skyline of the Bay, the Bay Bridge, and panoramic views of San Francisco.

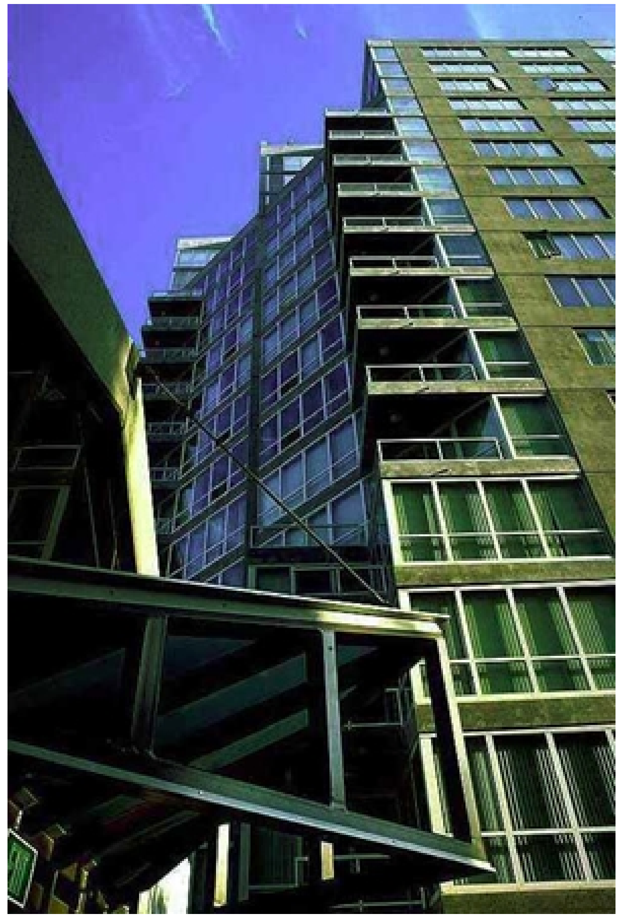

Fig. 4. A View of Avalon Towers
(Courtesy: Alexander Georges)

Form and Space. The building was designed as an elongated hexagon shape with six units per floor. This design strategy provides each tenant with a corner unit with views in two directions. The building and apartments are organized to maximize the number of units that have views of the city and the San Francisco Bay. Corner aparatments are highly desired by tenants because light and views come from two sides - and San Franciscans love their views.

Fig. 5. Entrance to Avalon Towers
(Courtesy: Alexander Georges)

The bedrooms are large (13'-6" x 13'-6") which allows for a computer desk or other working area in the bedroom. A bedroom of this size allows enough space so that this room can be multi-functional instead of just being occupied and filled by only a bed.

The building has special sound-insulating double-glazed windows to keep out the noise of the city. Each room has a balcony door or a window that can be opened to let in fresh air. The white window frames compliment the gray concrete of the building.

Contemporary architecture has often denied the quality of ritual and ceremony that has always been a part of religious and cultural life. In 388 Beale Street the curved entry form, the tall entry pavilion, the plaza, and the sequence of garden, fountain, and grand ramped stairs and promenade establish a sense of ceremony and serenity for the residential site.

Having one's own outdoor space is very important so that each tenant can step out of the door and get fresh air, grow outdoor plants, or barbecue on the balcony. The triangular balconies cantilever 6 ft. from the building at their furthest point. The balconies are designed to give the same feeling as riding in the bow of a ship.

Where people enter the building on the Beale Street side, a curved form intercepts the box to soften it and welcome pedestrians and cars. The building lobby is a 5-story vertical tower that immediately lets people know that they will be transported up to the plaza level. A 40 ft. glass art piece by Dan Winterich is installed on the glazed wall facing the street to filter out the direct view of the mundane 8-story post office building across the street.

Structure. One of the greatest pleasures people can derive from experiencing architecture is the appreciation and comprehension of how a building comes together with real materials; the comprehension of the building process.

The building is a concrete moment frame with the concrete frame exposed and expressed on the exterior of all sides of each tower. Everything that is not structural is infilled with white, aluminum-framed glass. There are no false or infill partitions on the exterior. The concrete is naturally colored and exposed and, in this state, expresses strength and durability. It soars, cantilevers, and steps upwards to twenty stories. The Towers step at various levels creating balconies and terraces for the units. Daring triangular balconies cantilever from the structure. These terraces provide the residents fresh air and fantastic views of San Francisco, the Bay Bridge and the Bay.

Many of the tenants work for Internet companies, and they have started a new wave in the city by insisting that their offices in existing buildings have exposed concrete walls, ceilings, etc. The building, designed to appeal to young tenants, incorporates natural materials throughout.

Conclusions

By 1919 the days of sailing ships were changing the character of the San Francisco waterfront and the heavy industry continued to ebb over the bay to Oakland. The area that seemed seedy in 1919 looked even more so in 1940. With the construction of the Embarcadero Freeway that was built as an overpass into the city of San Francisco in the 1960s, the area continued to remain an unattractive place. The earthquake of 1989 was the demise of the freeway, and perhaps this was the true beginning of rebirth. The accessibility of the freeways remained, but the visual aspects that were unattractive were removed and the neighborhood seemed free to breathe once again.

Few poets ever wrote about the glories of the South of Market Area – although one well-known author, Jack London, was born in the neighborhood. He wrote an essay entitled "South of the Slot" in 1909 in his book The Strength of the Strong. He describes the slot as a physical and social barrier between the working class and upper class neighborhoods:

"Old San Francisco was divided midway by the Slot. The Slot was an iron crack that ran along the center of Market Street and from the burr of the ceaseless, endless cable that was hitched to the cars, it dragged up and down. North of the Slot were the theaters, hotels and shopping district, the banks and the staid respectable business houses. South of the Slot were the factories, slums, laundries, machine shops, boiler works and the abodes of the working class. The Slot was the metaphor that expressed the class cleavage of society."

The computer industry has redefined the South of Market Area as new development has taken place in the 1990s. In a sense, the cleavage is just as great as it was in the nineteenth century.

Henry George in the periodical *The Overland Monthly* (1869), gave this succinct account of California, and certainly the San Francisco area was included (Olmsted, Roger R. and Nancy L., 1979):

"What constitutes the peculiar charm of California which all who have lived here long enough feels? Not just the climate alone…in California there may have been a certain cosmopolitanism, a certain freedom and breadth of common thought and feeling natural to a community made up from so many different sources in which every and woman has been transplanted with the native angularities of prejudices and habit more or less worn off."

It would seem that this spirit continues, and this makes the urban landscape of change attractive for those who seek its refuge.

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Building Where They Said It Couldn't Be Done

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Abstract

Construction of tall buildings and other major structures in large cities often involves working within severe site-specific limitations that can give structural engineers an opportunity to make a decisive contribution to the economic feasibility of such projects. They may conceive innovative transfer systems, as illustrated by several unusual load-transfer challenges and solutions from the author's practice experience. Examples include: a building supported on a grid of girders above an active rail yard and suspended, in part, from cantilever trusses on the roof; twin towers cantilevered 10 meters over a trading hall using a shear-panel transfer system; tall buildings linked structurally to act as a single unit, resulting in major cost savings; a subway station built under a multi-level parking garage using a method that reduced cost and permitted the garage to remain in service throughout construction.

Morton International Building

Introduction

Construction of tall buildings and other major structures in large cities almost invariably involves working within severe site constraints. These can involve all aspects of architectural and engineering design. As will be illustrated with examples drawn from the author's practice in Chicago, creating opportunities for the development of "impossible" sites through innovative design represents a unique – and uniquely rewarding – challenge to the structural engineer.

Site conditions can result in a building having a structurally inefficient and irregular shape (e.g., it may have to be very slender). Such conditions can create a situation where there is no clear and direct path along which to transmit structural loads into the ground from floors located where they are functionally most desirable. It is the latter situation that is the primary subject of this paper. Some of the main classes of solutions to the problem of transferring structural loads to the ground along indirect paths will be outlined. This will be followed by a discussion of a few unusual load-transfer challenges and solutions drawn from the author's practice.

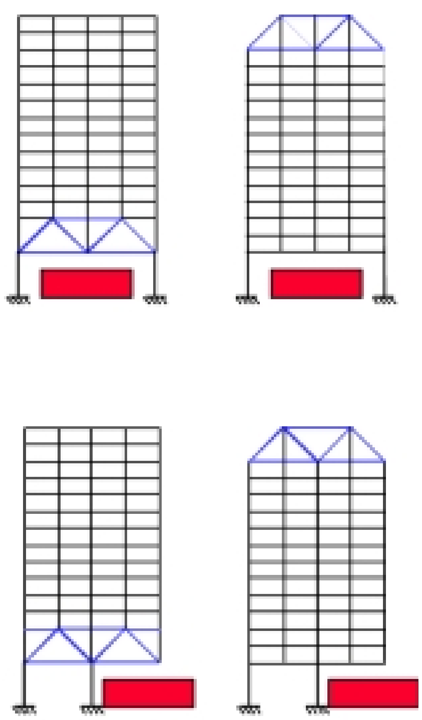
Types of Structural Transfers

At the most basic level, structural load transfer systems can be classified according to the type of load that is to be transferred – vertical, horizontal, or overturning.

Transfer of Vertical Load

When the direct downward transfer of vertical load to the ground is prevented by an obstruction, the solutions include spanning across the obstruction or cantilevering out over it, as illustrated in Figure 1(a). The transfer trusses or girders (trusses shown) could be located near the base of the building or at the top or anywhere in between. Locating the trusses or girders at the base will usually result in a simpler construction sequence and lower cost.

A possible alternative to the transfer cantilever shown in Fig. 1(a) is the tied-back shear panel transfer system shown in Fig. 1(b). In this design, vertical load is shifted laterally by means of a vertical wall panel (or diagonally-braced truss panel) loaded in essentially pure shear. A tie at the top of the panel and a compression strut at the bottom, both connected to the building's main lateral load-resisting system, restrain the corresponding moment.



Cantilever Over Obstruction

Fig. 1a, Transfer concepts for vertical load

Transfer of Horizontal Shear and Overturning Moment

Structural design concepts for transfer of horizontal shear and overturning moment from one part of a building structure to another are illustrated in Fig. 1(c). In the illustration on the left, shear alone is transferred, while the moment continues down to the ground. This type of transfer is usually a simple matter. Building floors are typically very stiff and strong in their own plane, and can be designed to transmit large in-plane forces at little additional cost. In the illustration on the right, both horizontal shear and overturning moment are transferred. The moment is transferred as a horizontal couple, using floors to transmit the horizontal forming the couple.

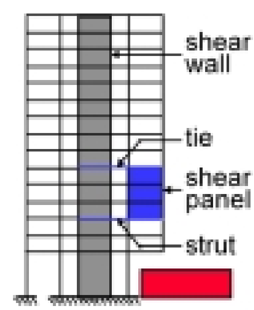
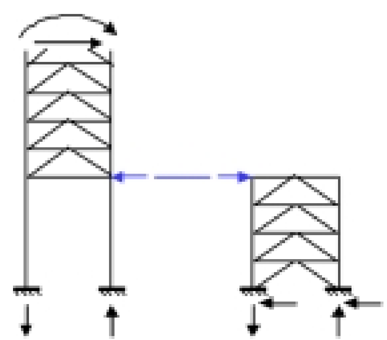
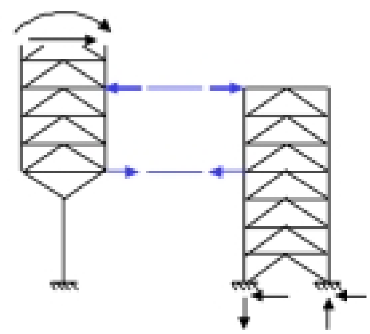


Fig. 1b, Tied-back shear panel transfer concept for vertical load



Transfer of Shear Alone



Transfer of Shear and Moment

Fig. 1c, Transfer concepts for horizontal shear and overturning moment

Applications

The use of innovative structural transfer systems will be illustrated with five examples drawn from the author's consulting engineering practice. Four are tall building developments; one is a subway station. One of the projects was not actually built; it succumbed to changes in market conditions late in the design process. The other four examples, all in Chicago, are projects that have been completed. In the following discussion of the five structures, some simplification and idealization of actual conditions will be made for purposes of clarity. The discussion is centered on the transfer systems. (Basic information on the three completed tall buildings can be found in the CTBUH database.)

Morton International Building

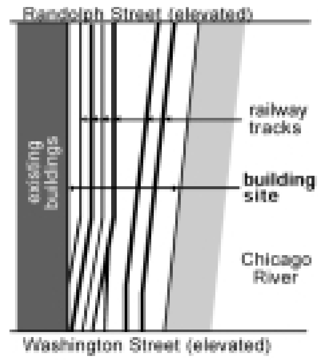
The Morton International Building (Dixon, 1991), at 100 North Riverside in Chicago, is a 36-story, 101,000 m2 office building. The lower 12 floors, each 4,300 m2, hold lobbies, parking space for 435 cars, and a 26,000 m2 computer center for the local telephone company. The upper floors hold rental office space.

The entire project is above an active rail yard, which had defeated all previous attempts at developing the site, though it is at a prime location on the Chicago River. As shown in the schematic site plan in Fig. 2, the rail tracks cover almost the entire site. The streets in the area are about 10 m above the tracks.

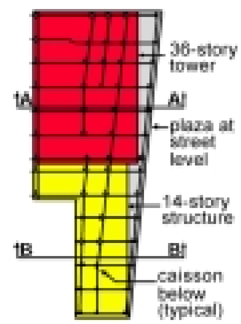
Development of the Morton International site was made possible by a comprehensive transfer system. Foundation caissons (drilled piers) and track-level columns were located where track clearances were adequate, as shown in the caisson grid in Fig. 2. Because of the irregular track layout, the caisson locations could not coincide with column locations in the building above (see superstructure grid in Fig. 2).

A complete grid of concrete transfer girders, about 2.5 m deep and between 1.0 and 2.5 m wide, transfers load from the building columns above to the track-level columns and caissons below. The top of the girder grid is at street level. Schematic Section A-A in Fig. 2 shows the relationship between building columns, transfer girders and caissons.

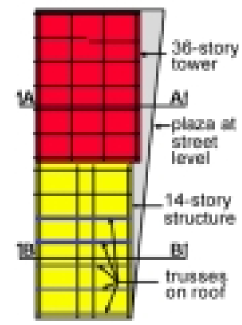
As shown in the caisson grid in Fig. 2, there was no room for caissons or columns among the tracks in a 20 m x 46 m area at the southwest corner of the site (north is oriented toward the top of the site plan). In early planning concepts, this area was left unbuilt. However, the telephone company demanded full 4,300 m2 floors; efficiency of the parking layout also required full floors, without a cut out in the corner. The solution was to provide a cantilever transfer system to support this part of the building. Cantilever trusses at the bottom, just above the tracks, would have been most economical but would have disrupted the parking layout. So the trusses were placed on the roof, where they became part of the architectural expression of the building, as indicated in Section B-B and the photograph in Fig. 2.



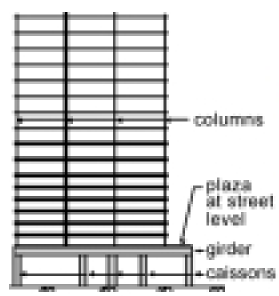
Site Plan



Cassion Grid



Superstructure Grid



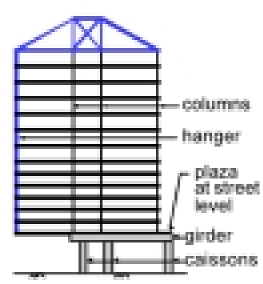


Fig. 2. Morton International Building

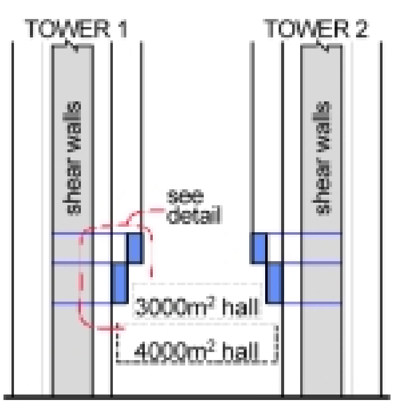
Chicago Mercantile Exchange Center

The Chicago Mercantile Exchange Center (Architectural Record, 1983), at 10 and 30 South Wacker Drive in Chicago, includes two 40-story, 116,000-m2 office towers and two stacked column-free trading halls, of about 4,000 m2 and 3,000 m2, respectively. The photograph in Fig. 3 shows the two towers and the structure enclosing the trading halls.

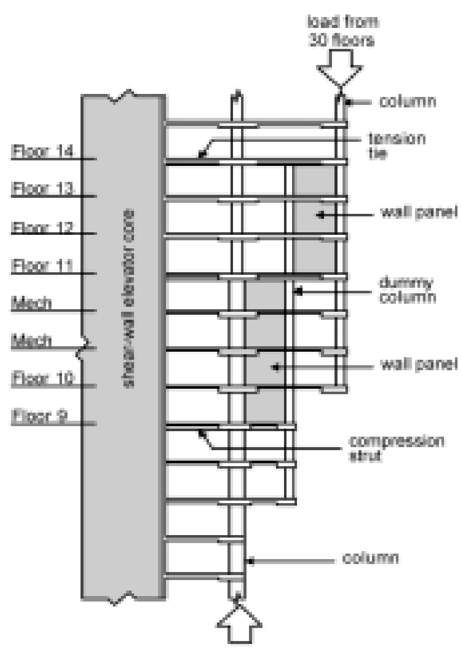
Typical floors in the office towers are of just under 3,000 m2, an area considered optimum in the local office leasing market. The challenge to the structural engineer on this project was to accommodate two-3,000 m2 office towers and a 4,000-m2 column-free trading hall on a site with a total area under 8,000 m2. The innovative solution was to cantilever each tower about 10 m over the trading hall, as shown in the schematic elevation in Fig. 3.

The cantilever was achieved using the tied-back shear panel concept, as shown in Fig. 3. The 10-m horizontal transfer was realized in two steps over seven stories (with a total height of 25 m). The shear panels are reinforced concrete walls 760 mm thick. The tension tie at the top and the compression strut at the bottom transfer overturning moment in the form of a horizontal couple to the shear wall core. The moments imposed by the transfer system cause lateral deformation of the shear core. The towers were erected out-of-plumb by up to 100 mm to compensate for this. Subsequent lateral displacements, including long-term effects, brought the towers to a plumb condition.

Chicago Mercantile Exchange Center



Schematic Elevation



Core Section

Fig. 3. Chicago Mercantile Exchange Center

Unbuilt Mixed-Use Project

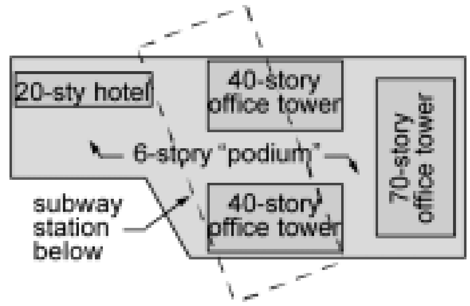
This example deals with a very large mixed-use project that involved extensive transferring of both vertical and lateral load. The general layout of the project, simplified and idealized for clarity, is indicated in Fig. 4(a). It includes a 70-story office tower, two 40-story office towers and a 20-story hotel, with a common 6-story base or podium holding retail space. Parking is accommodated in several below ground basement levels.

A subway station running diagonally across the middle of the property (Fig. 4) had discouraged all previous attempts at developing the site, even though it was at a prime location. The solution was a grid of cast-in-place concrete transfer girders just above the station roof slab. The concept is similar to that adopted for the Morton International Building (Section A-A in Fig. 2), but without the cantilevers and hangers.

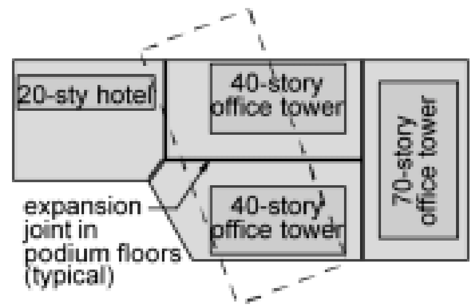
Early designs for the project included expansion joints through the 6-story "podium" structure to separate it into four structurally independent segments, one at each tower, as indicated in Fig. 4(b). The office towers used braced-frame cores with outriggers to supercolumns as their lateral load-resisting systems. The diagonal bracing could not be carried down through the lower six stories since the retail space below the tower cores needed to be open; so massive steel rigid frames were proposed in the podium floors below the cores.

In the final structural concept, as illustrated in Fig. 4(c), the expansion joints were eliminated, causing the entire four-tower project to act as a single structure. Therefore, a separate bracing system was not needed below each tower in the podium floors. Bracing and walls were provided wherever they would fit conveniently, scattered throughout the complex, below Level 7, as shown on the left side of Fig. 4(c). The slab at Level 7 was designed to transfer horizontal shear forces from the tower bracing systems to the podium bracing as shown on the right side of Fig. 4(c).

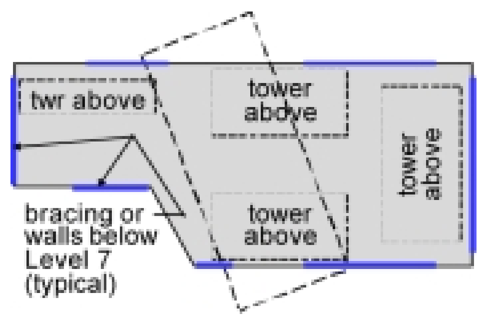
The redesign to eliminate the expansion joints and transfer lateral loads as indicated in Fig. 4(c), together with a few other structural refinements, reduced the estimated cost of this project by $60 million.



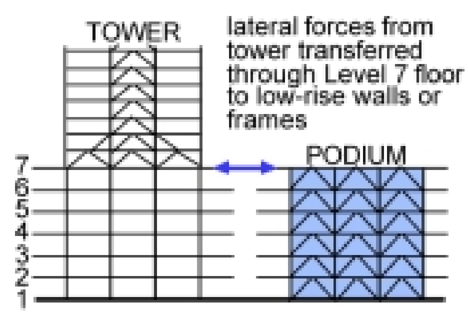
a. Overall Layout of Project



b. Conventional Expansion Joint Configuration



Shear Walls and Bracing in Podium



Lateral Force Transfer Concept

c. Design Concept with no Expansion Joints

Fig. 4. Unbuilt Mixed-Use Project

Boulevard Towers

The Boulevard Towers office development, at 205 and 225 North Michigan Avenue in Chicago, consists of a 44-story, 86,000 m2 South Tower and a 24-story, 82,000 m2 North Tower. Up to the 19th level, a 30 m wide infill structure spans the 12 m space between the two towers, resulting in floors of over 5,600 m2 each.

The structure is constructed of reinforced concrete, with shear wall cores as the lateral load-resisting system.

The 19-story infill between the two towers (see Fig. 5) links the towers structurally, to make them act as a single unit. This eliminated the need for expansion joints, which would have been subject to very large relative movements – of the order of 300 mm at the 19th floor – which would have been difficult to accommodate in the architectural and functional design of the project.

Moreover, use of the infill floors to link the two towers structurally offered important benefits. The lower North Tower has much larger floors than the taller South Tower. (Typical floor areas are 3,200 m2 in the North Tower and 2,100 m2 in the South Tower.) Architectural and space-planning requirements made it possible to have a deep shear core in the stubby North Tower, but only a shallow core in the slender South Tower. Linking the towers (see schematic elevation in Fig. 5) allowed the deeper, stiffer core in the lower building to resist most of the combined lateral loading imposed on the two towers.

The link floors represent a transfer system for both shear and moment, as shown schematically on the right side of Fig. 1(c), except that not all the moment is transferred from the taller to the shorter tower. At the base, the two shear cores share overturning moment roughly in proportion to their stiffness, with the core of the lower building supporting significantly more than half the total.

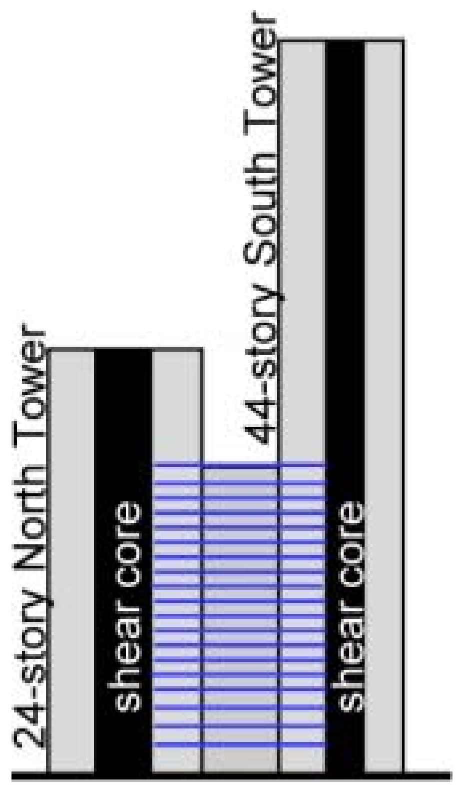
Compared to a design with the towers separated by expansion joints, the linked design of this project resulted in only a modest additional cost in the shorter tower and major savings in the cost of the taller tower.

Subway Station at O'Hare Airport

The extension of a subway line to Chicago's O'Hare Airport required the construction of a new station under an existing six-level concrete parking with structural spans of just under 20 m. To accommodate the station, which had to be column-free, the caissons (drilled piers) supporting two rows of existing garage columns had to be removed. Each pair of these existing columns is picked up by a 2.5 m wide x 3 m deep post-tensioned concrete transfer girder supported on new columns at the edges of the new station. The photograph in Fig. 6 shows the interior of the station; the girders are visible at the top.

The most innovative structural aspect of this project was not the configuration of the finished structure, but rather the sequence of construction and related structural design features. This permitted the entire garage, includ-

Boulevard Towers



Schematic Elevation

Fig. 5. Boulevard Towers

Fig. 6. Interior of Station

ing areas directly above the station, to remain in service throughout the construction period.

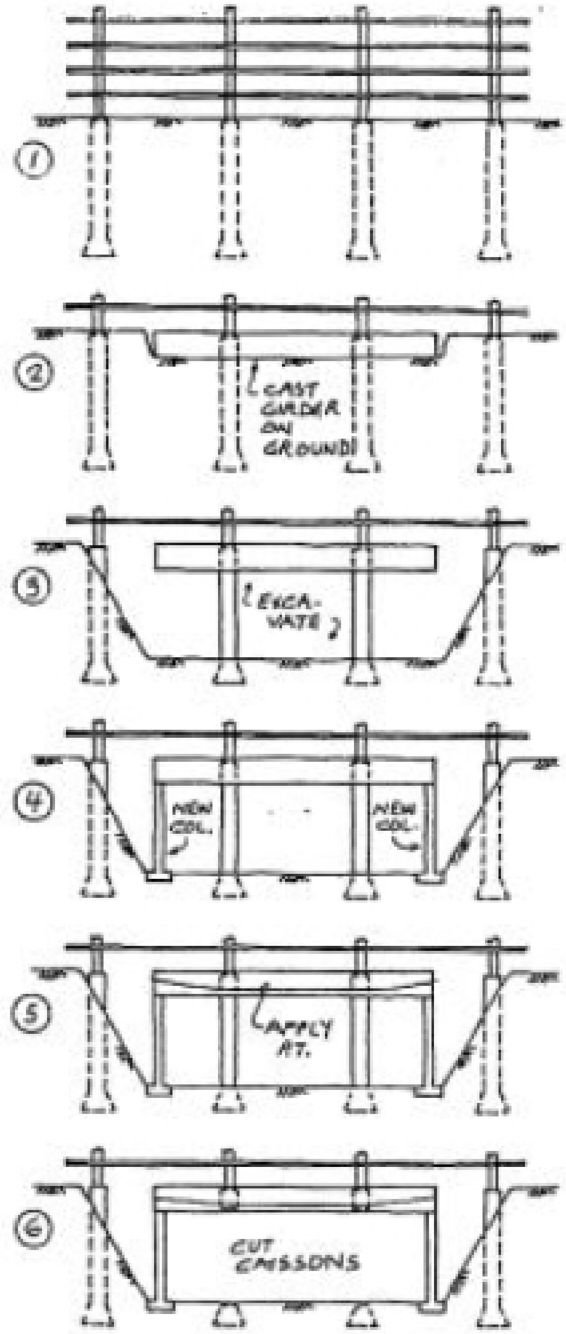
The construction sequence is indicated in Fig. 7. The transfer girders were cast on the ground, before excavation of the space for the subway station (Step 2). This yielded major savings compared to construction on shoring up in the air after excavation. The girders were fastened to the existing caissons and were supported by the caissons during excavation (Step 3) and the construction of new columns (Step 4). Post-tensioning was applied (Step 5) to transfer column reactions from the existing caissons to the new girder and columns. The caissons were then removed (Step 6) and the interior of the station was completed.

Conclusions

As cities become ever more densely developed, structural engineers will have increasing opportunities to make decisive contributions to the economic feasibility of future projects by conceiving innovative transfer systems using basic concepts of statics.

Innovative structural transfer systems can create opportunities for development where they didn't exist before. The Morton International Building and the Chicago Mercantile Exchange Center illustrate this, as well as the unbuilt mixed-use project discussed in this paper. Combining a transfer system with an innovative construction sequence can offer special cost and functional benefits, as illustrated by the design and construction of the O'Hare Airport subway station.

Besides carrying gravity loads to the ground along indirect paths, structural transfer systems can also be used to transfer lateral loads from one part of a project to another. Transfers of this type can produce major reductions in the cost of the overall development, as illus-



Construction Sequence

Fig. 7. Subway Station at O’Hare Airport

trated by the unbuilt mixed-use project and the Boulevard Towers buildings, where the transfer systems resulted in the integration of towers into a single structure that would have otherwise behaved as independent structural units.

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Complete Retrofit of a 47-story Steel Building for Wind Loads

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Abstract

This paper describes the retrofitting of an existing 47-story all-steel building with four basements in Houston, USA. Exterior steel spandrel beams welded to columns 30 ft. (9.1 m) on-center provided the original lateral resistance. The original structure was designed for strength based on wind loads of the 1971 building code. In 1994–95, it was retrofitted for the latest building codes and industry practice.

Introduction

In the late 1980s a major international company in Houston, Texas began searching for a location having approximately 1,000,000 square feet (83,000 m^2) to consolidate its headquarters staff and those of various subsidiaries scattered throughout the city. Location and program concerns led the client, a national real estate company, to seriously pursue large existing buildings in the downtown area. Research showed a significant number of piecemeal spaces but very few buildings available in their entirety. The search quickly centered on a 47-story building in use as contract office space, with over 50 tenants in residence.

Initial discussion soon identified major problems. The building was severely under-structured and covered with asbestos spray-on insulation. The previous owners of the building were in the early stages of abatement and planned to add a massive concrete core wall to increase its strength to meet current building code requirements. Investigation of this solution for the new client indicated a lack of future flexibility in mechanical and electrical penetrations at the core walls. There were also questions concerning its torsional rigidity for eccentric wind loads. These issues prompted evaluation of the structural requirements and possible solutions consistent with the projected use of the building.

The following presentation details the project and the solutions to various challenges presented by the existing structural system and the needs of the client.

Project Description

The project is the retrofitting of an existing 47-storyall steel structure (with four basements) built in Houston in 1971. The floor-framing plan of the original building is shown in Fig. 1. Welding the exterior spandrel girders to the exterior W14 columns, spaced at 30 ft. (9.1 m) on center, provided the lateral resistance of the building. The original structure was designed for strength based on the wind loads required by the prevailing building code which were much lower than the current code. It became apparent that the sway of the building and the P-delta moments were not considered in the original design. The building was purchased in 1994, and it was decided that the building should be retrofitted for the requirements of the corporation, the 1994 Houston Building Code, and industry practice. The building tenants were relocated elsewhere before the retrofit construction began.

The following major structural modifications were required:

a. The building was to be redesigned for the upgraded wind loads of the 1994 Houston Building Code and a force balance wind tunnel test.

b. The building was to be redesigned for current sway criteria and ISO motion perception limits.

c. A new 87 ft. (26.5 m) tall "hat" was to be added above the existing roof.

d. New lobby space was to be created on Floors 1 to 4. This involved the removal of one complete floor, two partial floors, and the upgrading of other floor areas.

Six schemes were examined for the structural retrofit of the main wind-resisting frame of the building. The system finally selected was to make a total of eight of the existing exterior steel columns into composite "super-columns" with 9-story tall diagonal steel bracing between these columns above Floor 5 (Figs. 2 and 3). The exterior corner bays did not have diagonals due to tenant considerations. As shown in Fig. 3, the diagonals were placed on the three bays of each exterior side. Below Floor 5 all exterior columns down to the existing mat were made composite. This composite frame carried the wind loads to the foundation as shown in Fig. 3.

Technical Issues

Wind Engineering. The original building was designed for strength in accordance with the old Houston Building Code. This code prescribed a wind pressure of 20 psf (1.0 kPa) up to a height of 60 feet (18 m) and 30 psf (1.5 kPa) above that height. Under this code the base shear was 2,830 kips (12,578 kN) and the base overturning moment was 915,000 kip-ft. (2,620,000 kN-m). Hurricane Alicia went through downtown Houston in 1983, and this caused an upward revision of the code wind loads. For the 1994 code loads the base shear is 5,100 kips (22,700 kN), and the base overturning moment is 1,930,000 kip-ft. (2,620,000 kN-m).

Since the structure was to be strengthened as indicated above, and a hat was to be added at the top, both the building's mass and stiffness were being modified. A decision was made to perform a force balance wind tunnel test. The analysis was based on the following fundamental periods of the strengthened building:
$T_x = T_y = 4.6$ secs.; and $T_{THETA} = 2.7$ secs.

The results of the wind tunnel test gave a maximum base shear of 3,490 kips (15,511 kN) and a maximum resultant overturning moment of 1,590,000 kip-ft. (4,552,787 kN-m). These values were used in the building design. It can be seen that the wind tunnel test results are higher than the original code forces, but were lower than the forces allowed by the 1994 code.

The force balance test also indicated a resultant acceleration at the top floor of 23 milli-g's based on a 10-year recurrence interval for 1.25% damping. This is within the ISO guidelines. The building has been instrumented with an accelerometer at the top floor. A low-level wind- storm was recorded on the instrument that gave a reading of the fundamental period of the building as approximately 4.5 seconds, which agrees well with the calculated value of 4.6 seconds.

Foundation Review. The choice of the retrofit scheme created some changes in the loads on the foundation. The use of composite columns increased the dead load on those columns. The wind analysis for the upgraded structure based on the new wind loads also substantially increased the axial loads on the composite columns and reduced the percentage of axial wind load carried by the corner column (Figs. 2 and 3) as compared to the original building design.

The original mat foundation of the building was 7 feet (2.13 m) thick. The critical checks involved the punching shear at the locations of the composite columns and the overall flexural design of the mat. In order to check the punching shear capacity, a total of 25 cores were taken in eight critical locations to obtain the in-place concrete strength. The lowest average in-place strength of the eight critical locations was used to check the adequacy of the mat. Furthermore, as can be seen in Fig. 3, the composite columns were widened at the mat level and a shear wall was added. A soil-structure interaction analysis was performed to verify that the existing mat was adequate without being strengthened.

Shrinkage and Creep of Composite Columns. The composite columns were built with concrete having a 56-day compressive design strength of 10,000 psi (68.9 mPa). This high strength concrete was used for three reasons: economics, higher modulus of elasticity, and reduced column size. Cost studies in the United States indicate that up to approximately 12,000 psi (82.6 mPa) a higher strength of concrete provides a lower cost per column. The modulus of elasticity was measured and was found to have a value of 5,800,000 psi (39.9 mPa). The reduction of column size is important because it minimizes the problems of tenant acceptance of the interior layout. Shear-studs were used on the steel column to assure good bond to the concrete encasement.

The placement of concrete encasement in a diagonally braced steel frame has two technical problems:
1) The shrinkage and creep of the composite column will induce stresses in the diagonals. It was desirable from a leasing and steel erection standpoint (where minimum weight is desirable) to keep the diagonals as small as possible. Hence, to minimize the shrinkage-induced stresses in the diagonals, the diagonal connections at the composite super-columns used slotted holes. The bolts were left loose until the super-columns had been completed for the whole building to allow much of the shrinkage (70%) to occur. The bolts were subsequently tightened. It should be noted that the connections of the diagonals to the steel columns were tightened in the normal sequence of construction.

2) The second challenge posed by the shrinkage and creep of the steel composite super-column is the potential problem of differential axial shortening between exterior composite super-column and interior steel columns. Since the existing interior steel columns are set, any shortening of the exterior makes it go down relative to the core.

Therefore, it was decided to use expansive cement in the construction of the exterior composite columns, and a test program was initiated. A testing laboratory was engaged to help develop the proper mix design. After discussion with the supplier of the Type K (expansive) cement, the laboratory prepared five trial batches with water to cementitious materials ratios varying from 0.26 to 0.37. Each batch used cementitious material of the following blend: 67% Type I cement, 14% Type K cement, and 19% flyash. Their 56-day strengths were plotted. A water-to-cementitious material ratio of 0.30, which was found to produce a laboratory strength of 11,500 psi (79.2 mPa), was chosen for field use.

The testing laboratory performed modulus of elasticity tests, restrained expansion tests, shrinkage tests, and creep tests on the design mix. The restrained expansion tests indicated an average expansion of 0.044% after seven days in limewater. The shrinkage tests indicated an average expansion of 0.0187% still remained after 28 days of air curing. The expansion with time was found to be similar to that found in the literature about expansive cements. Surveys of floor levelness for the top occupied floor taken before and after the completion of the concrete super-columns did not show any significant axial shortening of the super-columns.

A drawing of the construction of a composite column is shown in Fig. 4. It should be noted that the column bars were spliced throughout the height of the building with mechanical tension couplers, so that the length of bars was less than required if they had been lap-spliced. This allowed the bars to be transported in the existing building elevators.

Steel Column and Diagonal Details

The strengthening of the loaded steel members required special precautions for welded connections. Gusset plates had to be welded to the loaded exterior columns to make the diagonal connections. A complete welding procedure was established. The welding sequence was established to restrict the temperature rise of the existing steel. The maximum temperature was set at 650° F (343° C) at 6 inches (150 mm) from the weld location and was measured by heat indicating crayons. The sequence of welding of gusset plates was as follows:

Steel was heated to a surface temperature of 200° F (93.3° C) to drive out surface moisture. At this stage, only weld stringer passes were allowed (no weaving or wash passes). Each pass was well peened (except for root and final pass) and the first pass of each layer was against the column flange. Electrodes with a relatively high nickel content were used. Welds were post-heated or insulated to avoid rapid cool-down.

It should be noted that one of the advantages of composite construction is that the amount of welding is greatly

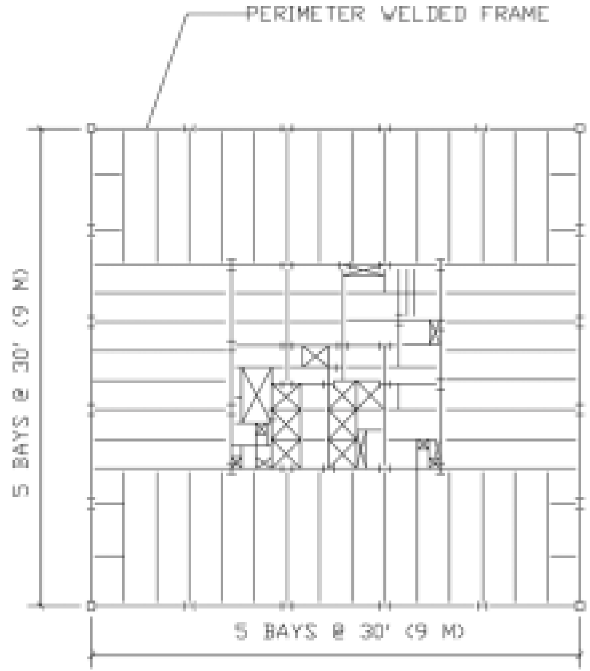


Fig. 1. Original Building Frame

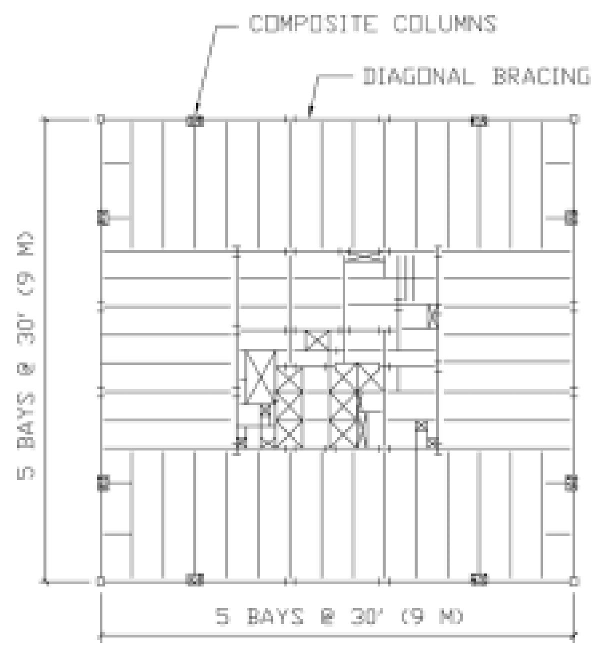


Fig. 2. Composite Retrofit

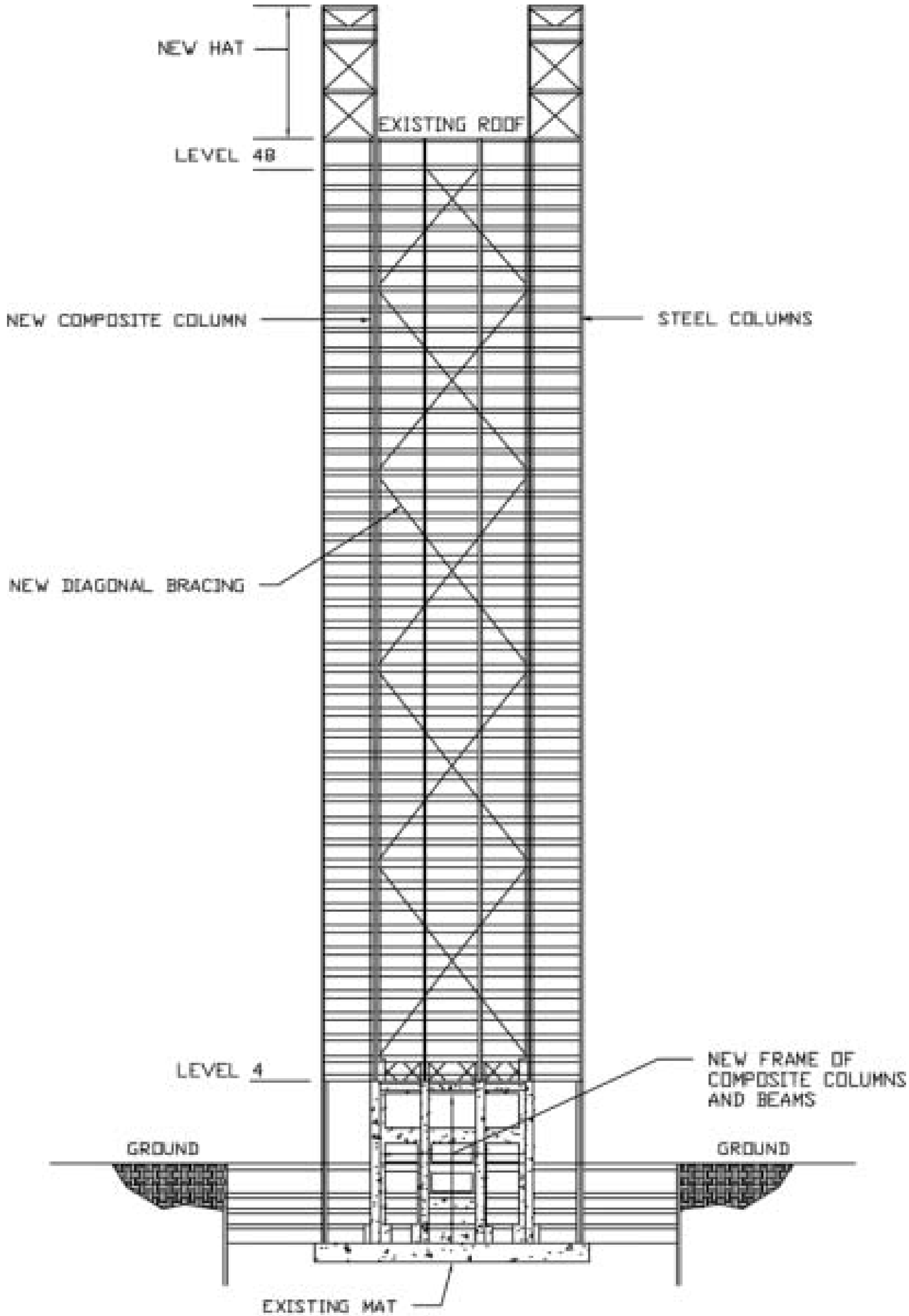


Fig. 3. Building Elevation

reduced thereby reducing risk and improving constructability.

Construction of Steel Diagonals. In order to minimize the weight of individual pieces which had to be lifted into the building, the steel diagonals consisted of four steel angles, the largest of which was 8 in. x 8 in x 1 in. (200 x 200 x 29 mm). Additionally, to facilitate construction it was decided to bolt the diagonal directly to the gusset plate. A drawing of a typical connection is shown in Fig. 5.

The gusset plates for the diagonals were installed according to the following general procedure. The floor was shored in the area of the installation of gusset plate. An opening measuring 3 ft.–8 in. x 1 ft.–6 in. (1.12 m x 0.46 m) was cut in the concrete slab, and the inside flanges of the perimeter spandrel beam were notched to the beam web. This allowed the placement of the gusset. The diagonals were loose-bolted to the gusset until three floors of gussets and diagonals were placed. After this three-floor set of gussets and diagonals was placed, it was aligned, accounting for the construction tolerances of the existing building. The gussets were then welded to columns and bolted to diagonals, and the shoring was removed.

The steel diagonals were not fire-proofed since their purpose was to increase the stiffness of the building under wind loads only. Adequate strength was available in the balance of the structure without the diagonals to meet the strength requirements of the building code.

Construction Considerations

Prior to the beginning of the retrofit, it was decided to remove the existing asbestos fire-proofing, which was eventually replaced with a cementitious fire-proofing. The asbestos removal sequence and project completion date required that the composite column work be done in three locations simultaneously. Work had to be done simultaneously from the mat to Floor 5, Floors 5 to 46, and Floor 46 to the roof.

This construction sequence required the careful consideration of two items. The exposed steel column had to carry the additional load of the concreted columns above. Moreover, detailed discussions were held with the contractor regarding the methodology of placing concrete in a column below when the column above was already concreted. The problem was solved by providing adequate reinforcing dowels and headed studs and providing air relief vents and a pressure head of concrete above the interface of the column above. A trial location at Floor 5 was selected, and the experiment proved to be successful and was used for all further construction.

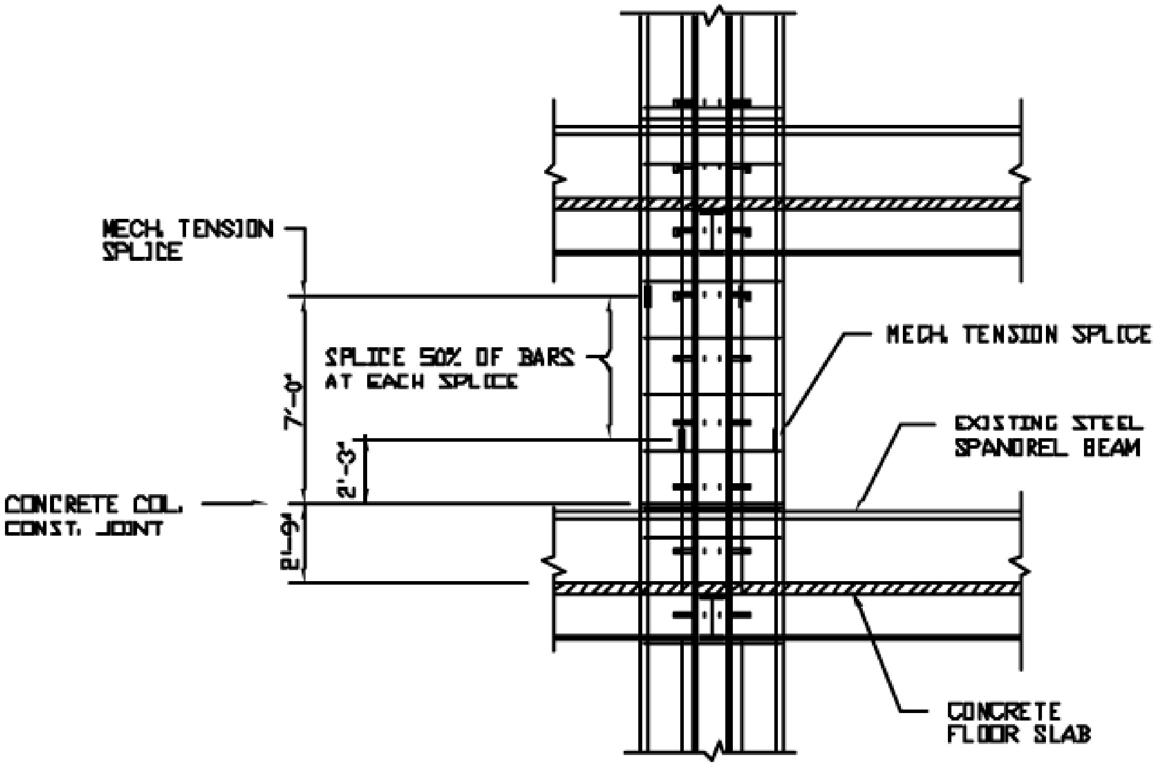
In order to create tall, dramatic entrance lobbies for the building, Floor 3 was removed from the building as well as portions of the Floors 2 and 4. Before these floors could be removed, however, two structural modifications were required. First, a horizontal in-place bracing-truss was added just below Floor 4 in order to properly brace the core columns to exterior lateral stability system of the building. Second, several of the core columns had to be cover-plated so that they would carry their loads with twice the unbraced length of their original design.

Security Concerns

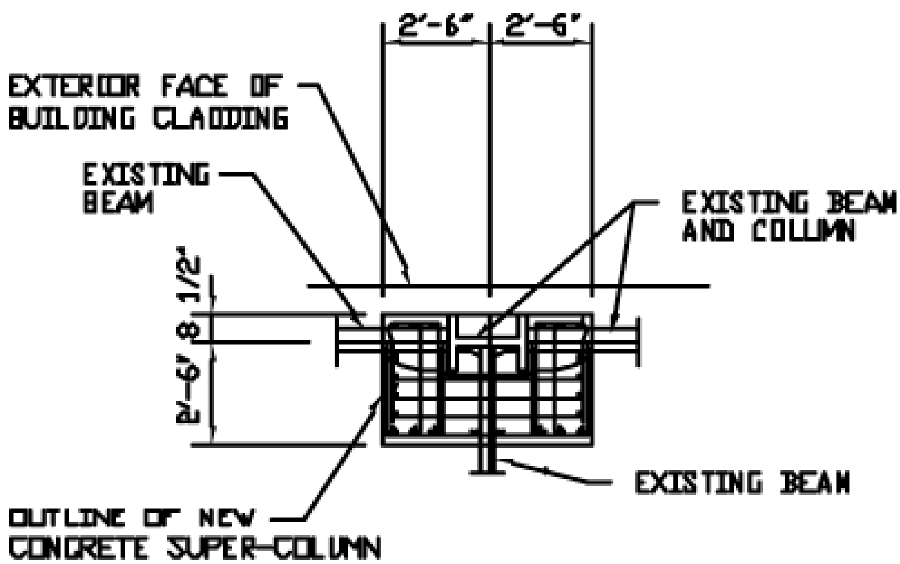
Very early on, the client expressed concern about the security of the building against terrorist attack. Several concerns were discussed about the structural security of the building and steps were taken to minimize the danger. First was the possibility of unauthorized vehicles in the basement loading dock, on the plaza level, or in the parking areas below the building. Second was the concern of military type cutting charges being used against exposed steel columns in the building. The approach to these concerns was to view the lower floors of the building as one type of security problem (e.g.: vehicle bombs) and the floors above Floor 4 as a second problem (e.g.: cutting charges).

The solutions included several architectural and security approaches that will not be discussed here. The structural solutions consisted of encasing the exterior columns in massive multistory walls of very high strength concrete in the parking garage, and blocking parking access to the area within the building floor plan. Composite columns extend upwards from the top of these walls (at one floor below grade) which are bridged at Floor 4 by a truss to aid in spanning across any column or wall section that has been compromised (Fig. 3). Above Floor 4, the eight composite super-columns, two on each face of the building, reduce the possibility of cutting charges being effectively used. Studies were conducted on the redundancy of the structure for localized damage. Computer simulation indicated the beneficial effect of the diagonals and the composite columns. If any of the exterior columns near the bottom of the building are removed, the tower still shall have a margin of safety against collapse.

The purpose of these changes is to strengthen the building against types of attack that historically have been used against buildings and steel structures around the world. While it is recognized that no building can be made totally impervious to attack, it is believed that structurally retrofitting existing buildings to minimize damage

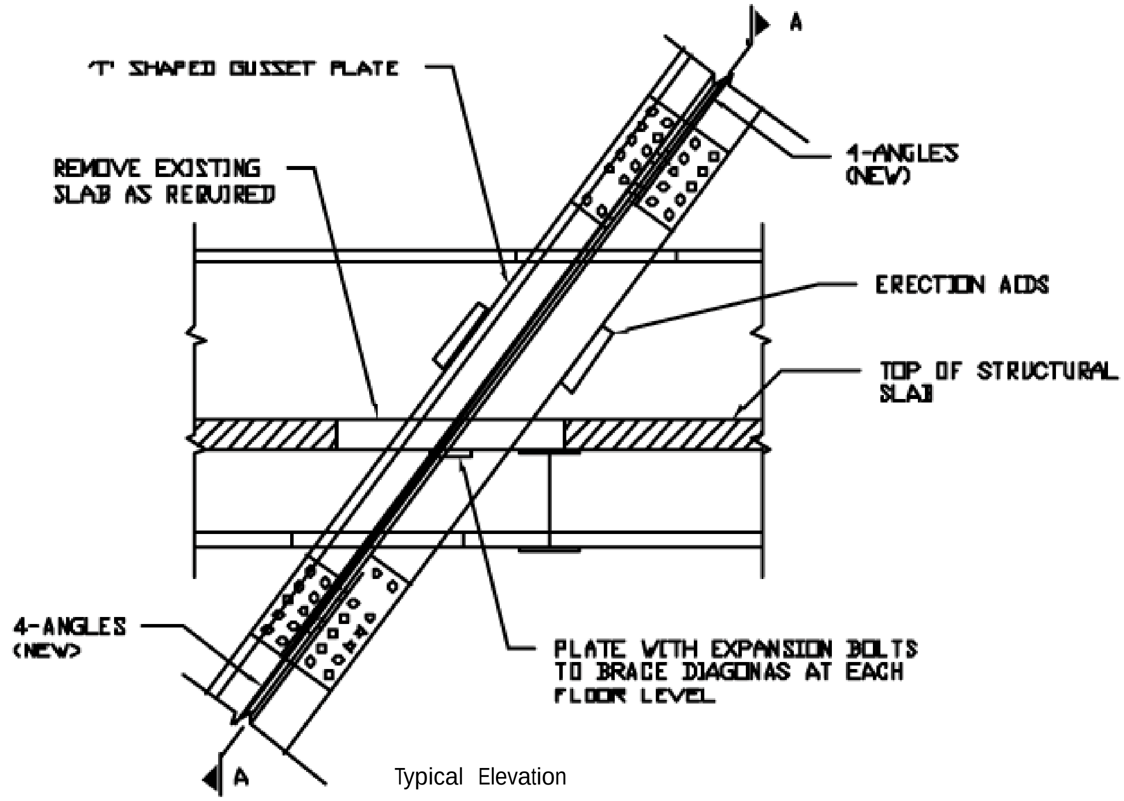


Elevation

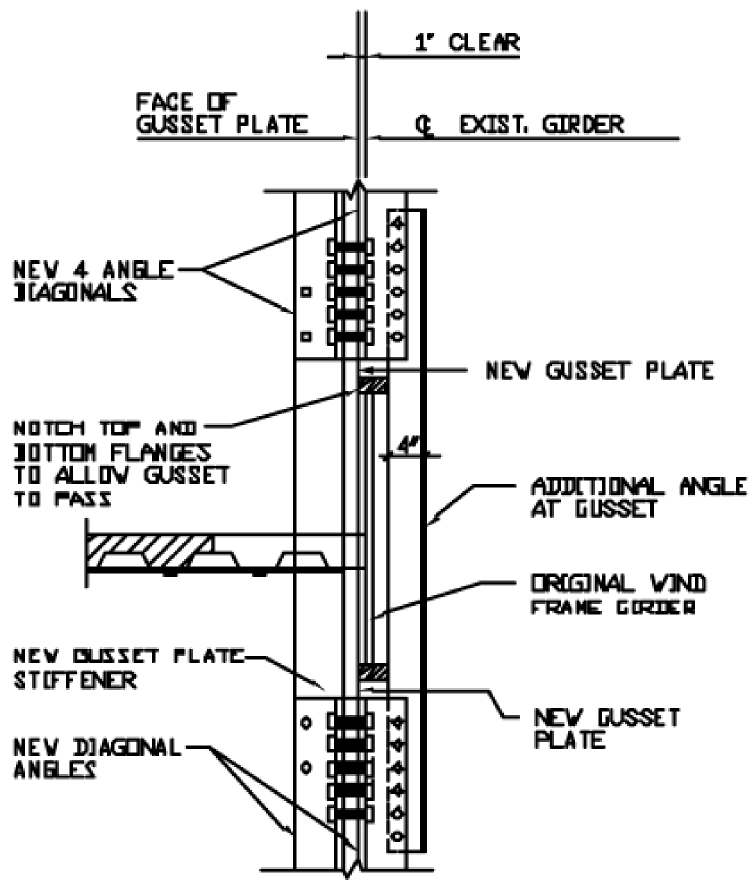


Cross Section

Fig. 4. Construction of a Composite Column



Typical Elevation



Typical Section A-A

Fig. 5. Typical Connection of Diagonals at Floor

can assist in deterring attack and greatly increase the possibility that attackers would fail in their ultimate objectives.

Conclusions

The retrofitting of buildings is a complicated task. The beneficial effect of using a composite retrofit for this building was amply proven. The composite retrofit increased the strength, stiffness, and damping of the structure. The concrete encasement of the existing steel columns also reduced the amount of welding required on a loaded column, thereby reducing the amount of welding required and reducing risk. The composite retrofit also enabled the construction to be done simultaneously at three floor levels reducing the construction time. Finally, the composite retrofit was economical, resulting in a saving of about 40% of the cost of other possible solutions. The total cost of the retrofit of the structure was approximately 15 million dollars. To date, it is possibly the tallest building retrofitted for wind loads.

Structural Design for Passive Seismic Control Utilization

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Abstract

A special steel moment frame building with supplemental damping devices was designed to achieve immediate occupancy performance levels for Design Basis Earthquakes (DBE). In order to protect the valuable contents in this building, seismic fluid dampers were chosen to reduce the lateral accelerations and displacements during earthquakes. The primary seismic frames, without dampers, were designed to meet the 1997 Uniform Building Code (UBC). The frames, without dampers, were designed to remain elastic under a DBE. The goal was to limit the lateral drift to 1% of the story height and the demand-to-capacity ratio of the moment connections within 1 under a DBE. Under this design criteria, buildings should sustain minimal or no damage to their structural elements and only minor damage to their nonstructural elements. In addition, business interruption after a major earthquake should be low.

Introduction

The addition of an Energy Dissipation System (EDS) in structures provides a new and innovative option to enhance seismic resistance. With the implementation of such a system, the demands on main structural elements would be reduced significantly during an earthquake. Currently, the UBC does not provide criteria for an EDS, but it does address design procedures for base isolation. Therefore, buildings with an EDS require specific design criteria and procedures. This study investigates steel moment frame structures, with and without seismic dampers, as they are subjected to various site-specific earthquake records. The EDS chosen for this study was a Fluid Viscous Damper because the damper forces are out of phase with axial loading of the columns, and the structural period does not alter due to the addition of fluid dampers.

Building Frames Description

A 4-story office building with supplemental damping devices was designed for better seismic performance. The beam members are typical W24 and column members are W27. The frame elevation is shown below in Fig.1. Approximately 20% of critical damping was provided. The brace with a damper and the brace connection are shown in Fig. 2. The gap between the gusset plate and the beam is designed to ensure that additional forces due to the dampers does not impact the existing axial forces on the beams. Because of the change from a moment frame to a braced configuration, load paths were altered to produce substantial axial loads in the columns. As a result, the number of piles and the size of the pile cap were significantly increased. Fig. 3 shows additional reinforcement that was added to compensate for peak column axial forces. As a result of the addition of seismic dampers, foundation design and construction become of significant concern.

Site Specific Ground Motion Records

Site specific ground motion records are required for time-history dynamic analyses for buildings with passive control utilization. Probabilistic hazard levels used in this design and their corresponding mean return periods (the average number of years between events of similar severity) are shown in Table 1. Response spectra should be developed for an equivalent viscous damping ratio of 5%. Additional spectra are also developed for other damping ratios. One pair of MPE and three pairs of DBE and MCE were prepared by geo-technical engineers. The time histories used in spectral matching are shown in Table 2, and the peak acceleration for level three earthquakes is shown below in Fig. 4. The time histories used in spectral and spectral accelerations at various damping ratios are shown below in Fig. 5.

Fluid Viscous Dampers

The simplest model to simulate the mechanical behavior of fluid viscous damper is the Maxwell model (Bird, et. al., 1987) given by where l is the relaxation time, Co is the damping constant at zero frequency, P is the damping force, and U is the damper position velocity. Constantionu & Symans (1992) and Pong, Tsai & Lee (1994) also investigated the mechanical behavior of fluid

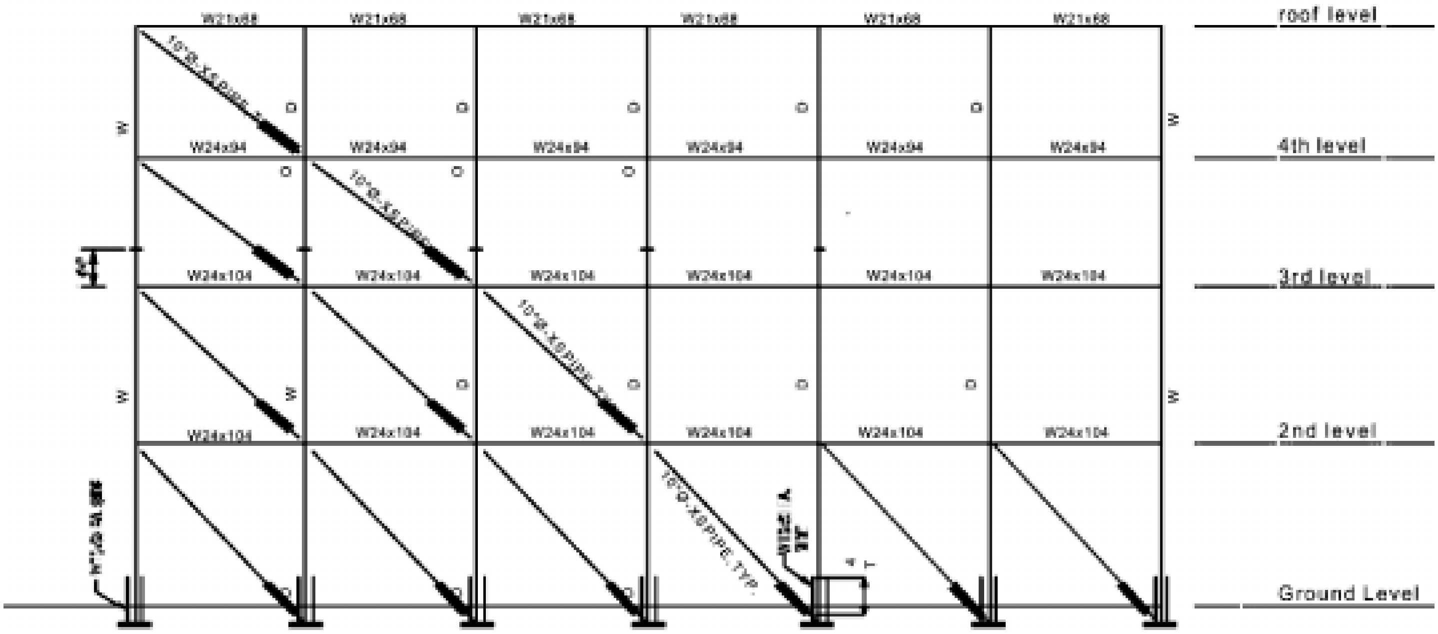

Fig. 1. Frame Elevation

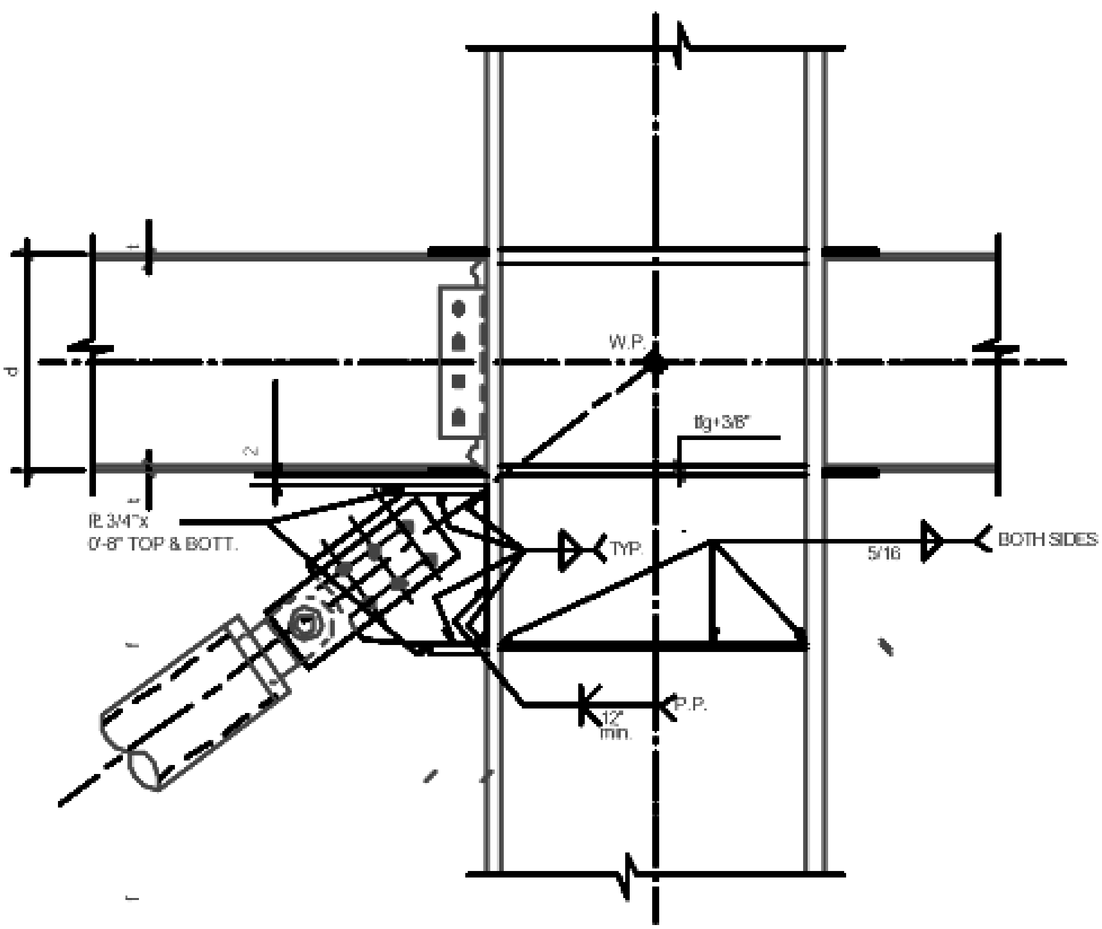

Fig. 2. Brace Connection

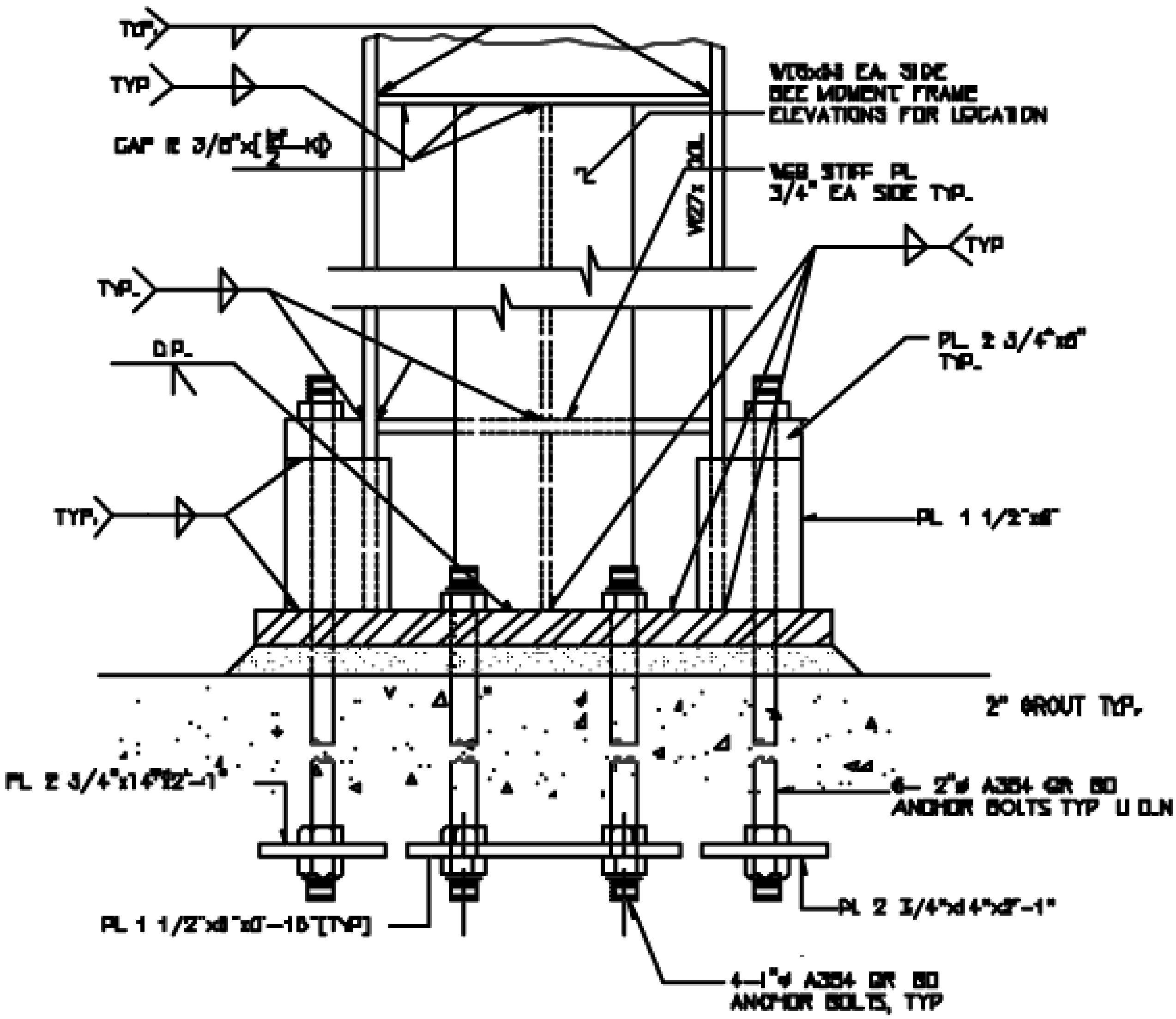

Fig. 3. Base Plates and Foundation

viscous dampers. Since fluid viscous damper force is a function of the damping constant and velocity, the damper force can be defined as F=CVa where C is the damping constant, V is the velocity, and a is a velocity exponent. The cyclic response of the damper is dependent on the velocity of motion. The mechanical behavior of the damper is dependent on the frequency and amplitude of motion (Constantionu & Symans, 1992). The addition of fluid viscous dampers should be detailed so that the period of the damped structure does not change in comparison to that of bare frames without dampers.

Design Criteria

Special steel moment frames without dampers were designed to conform to the UBC of 1997 requirements for strength and drift. Per the UBC static force procedure, the seismic design parameters are as follows.

The maximum inelastic response displacement under the UBC earthquake design force was limited to 0.02 times the story height. The structural period was found to be about one second. As a result, the static base shear force equals 7.2% of total story weight. The stress on members was found to be relatively small using the Load and Resistance Factor Design method. However, the final design of moment frame members was governed by the time-history analyses for the damped structure. Steel moment frames with dampers were designed to remain elastic under a DBE. The goal was to limit the lateral drift to 1% of the story height and the Demand-to-Capacity ratio (DCR) of the moment frame members and connections to within 1 under the DBE events. In case of a larger earthquake such as a MCE, plastic hinge formation and some structural damage would be expected. Therefore, the post-Northridge moment frame connections were designed to provide the ductility to minimize the structural and nonstructural damage in the event of a MCE. Reduced beam section connections

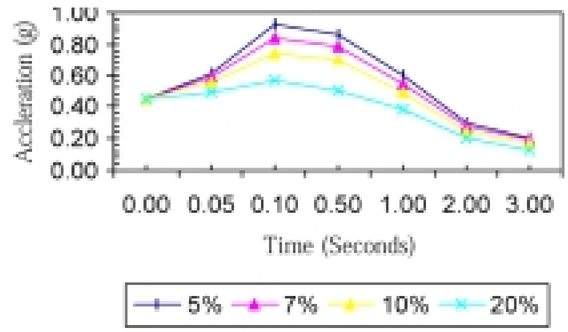

Fig. 5.
Response Spectrum at various damping ratios

Earthquake Definition	Earthquake Having Probability of Exceedance	Mean Return Period (years)
Lower Level Earthquake (MPE)	50%/50 year	72
Design Basis Earthquake (DBE)	10%/50 year	474
Max. Credible Earthquake (MCE)	10%/100 year	950

Table 1. Earthquake Definition

Design Earthquake	Earthquake	Magnitude	Time History	Dist. (km)
MPE	Loma Prieta	7.1	Santa Teresa	18
DBE	Imperial Valley	6.7	El Centro	12
MCE	Imperial Valley	6.7	El Centro	12

Table 2. Time Histories Used in Spectral Matching

(FEMA, 1995 and 1997) were selected to produce an intended plastic hinge zone. Steel moment frames with supplemental damping were designed to have a DCR of less than 1 under site-specific time-history dynamic analyses. Three DBE time-history analyses were performed and the maximum response of the parameter of interest was used for the final design. Each pair of site-specific time histories was applied simultaneously to the computer model, considering the most disadvantageous location of mass eccentricity (UBC 1997).

Seismic Zone Factor, Z	0.4
Important Factor, I	1.0
Site Profile Type	S_D
Ductility Factor, R	8.5
Seismic Force Amplification Factor, Ω_o	2.8

Diagonal braces were designed to ensure that DCR remains within 1 / 2 so that the braces would have higher safety factors. As a result, 10-inch (244-mm.) diameter extra strong pipes were selected to meet the design and construction requirements. Selecting stronger pipes ensures that the braces function properly under higher earthquake demands. Although the load paths are also changed, the seismic dampers can reduce the story drift and thus reduce the column bending moment. This alteration of load paths leads to substantial axial loads in the columns. As a result, demands on the foundation were significantly increased with the addition of dampers. More piles and larger pile caps were designed to keep the DCR to be less than 1 at strength level.

Structural Seismic Response

The DCR for beams at MPE, DBE, and MCE are tabulated in the Table 3. Figs. 6 and 7 show that both the story drift and story lateral acceleration are reduced significantly with the use of dampers during earthquakes.

Conclusions

Since the story drift ratio is limited to less than 1% during a DBE, the structural and nonstructural damage will be minimal. In the event of a MCE, the post-Northridge moment connections will yield to provide additional structural damping to reduce the demand on primary structural frames. More piles with increasing length and larger pile caps were designed to meet higher performance demands. In addition, collector forces were found to be much higher than originally expected. Therefore, larger connections and beam collector members were redesigned to meet this requirement.

Structural behavior is not much different during lower demand earthquakes such as a MPE or DBE, since the structure remains elastic under those events. However, structural behavior is quite different under a MCE because demands on the joints increase significantly. Table 3 shows that the DCR is larger than 1 under a MCE. Under MCE events, the primary structural members contribute larger energy dissipation, which reduces the role of the dampers as a major mechanism for energy dissipation. Due to the complexity of structural seismic response and the uncertainty of ground motion, the effects of inelastic behavior of frames with EDS become important. The effectiveness of seismic dampers is reduced when the structure experiences inelastic deformations. In such cases, force might be either increased or reduced when the structure is responding beyond its elastic range. The complexity of structural behavior during earthquakes makes it difficult to present a clear design

	MPE	DBE	MCE
Roof	0.58	0.78	0.93
4th Floor	0.66	0.92	1.00
3rd Floor	0.74	0.94	1.10
2nd Floor	0.75	0.99	1.14

Table 3. Demand / Capacity Ratio for beams

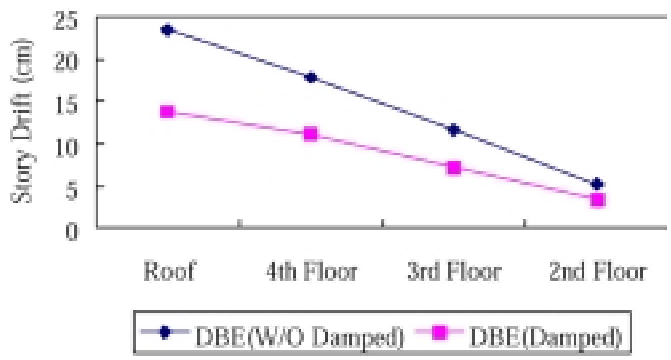

Fig. 6. Story Drift for Frames with and without Dampers at DBE.

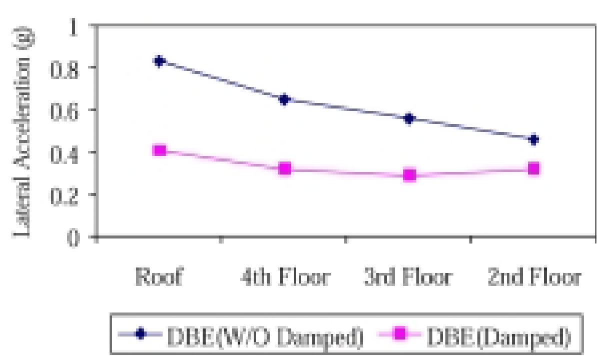

Fig. 7.Story Lateral Acceleration (g) for Frames with and without Dampers.

method for engineering professionals. Therefore, extensive verification of structural dynamic behavior is required.

In this study, the results show that the intensity of an earthquake is the primary factor affecting structural performance. For lower level earthquakes, such as a MPE, the structure has enough stiffness and strength to resist earthquake demands. Therefore, the addition of supplemental damping is relatively insignificant. At the DBE level, the added damping becomes a very important means of reducing structural response assuming the structure remains elastic. At the most severe level earthquakes, such as a MCE, the added damping may become insignificant if the structure undergoes inelastic action. Because of the characteristics of ground motion, such as its amplitude and frequency content, which will affect the amount of energy imparted to a structure, the time history analyses should be done wherever practical with many different ground motions. As a result, design parameters should be based on ground motion characteristics. It is also recommended that structural stiffness and strength are proportionally distributed to ensure structural regularity. Dampers should be arranged in such a way so that structural integrity and strength remain uninterrupted. Structural connections should be detailed properly so that they provide significant ductility in case of inelastic deformation during severe seismic excitations.

It is recommended that structural behavior is checked for a MCE. Under this earthquake level, the structural connections are likely to form plastic hinges and this will complicate the structural analysis.

Energy dissipation devices should be designed considering environmental conditions such as wind, fatigue, ambient temperature, operating temperature, and other damaging substances. A good quality control program should be implemented to ensure that the dampers consistently function properly after installation. A well-established maintenance document is a useful tool to ensure better long-run service. Supplemental damping devices are still new to many professionals. The seismic behavior of structures with dampers requires more research and testing to ensure reliability. Therefore, continuing research efforts and professional education and training are needed in this field.

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Reflections on the Hancock Concept

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Abstract

This paper reviews the stages of conceptual design through the final design of the John Hancock Center in Chicago. Several unique features of the project considered during the design process are discussed. The structural system introduced the braced-tube concept for the first time. The collaborative effort among the architects, engineers, and fabricators that made the project successful is emphasized. The building has withstood the passage of time for the last 30 years. It remains as an icon of Chicago and a symbol of structural expression in architecture.

Introduction

The John Hancock Center of Chicago (Fig. 1) was awarded the 25-Year Architectural Excellence Award by The American Institute of Architects in February 1999. This award is given to a building whose architecture has withstood the test of time and, after 25 years, still represents the excellence it originally had. It is actually a testimonial to the enduring quality of the architecture. In the case of the John Hancock Center the structural expression of the diagonalized trussed-tube constitutes the essence of its architectural esthetic. In this sense, the structure itself has withstood the test of time in the context of its architecture.

The conceptual design of the Hancock Center was started in 1965 and the building was completed in 1970. Today the Hancock Tower, with its powerful diagonalized expression, represents a unique icon for the City of Chicago. This paper is written to commemorate the 25-year award with an examination of the conceptual development of the system and its enduring legacy in tall building systems technology. It is also relevant to note the evolution and impact of computers on the design process since 1965 when the Hancock Center was conceptually designed. It is particularly relevant as this paper is being developed for the Internet, a new computerized medium.

The Sears Tower of Chicago, which until recently was the tallest structure in the world, represents another breakthrough in the systems methodology with respect to tall buildings. The Sears Tower was started in 1969 during the same time period as the Hancock Tower and involves a unique bundled-framed tube system. Hancock and Sears taken together represent a quantum jump in tall building structures in a short period of time. The comparatively rapid evolution from frame buildings to these unique high efficiency cantilever concepts will be examined here. The author was involved in the conceptual development and design of the Hancock Building. The paper contains the author's observations on this entire design process on the basis of this involvement.

Fig. 1. John Hancock Center of Chicago

System Evolution

Tall building systems as an organized structural system have a relatively recent history dating back to only the late nineteenth century. The Vierendeel form steel-tiered

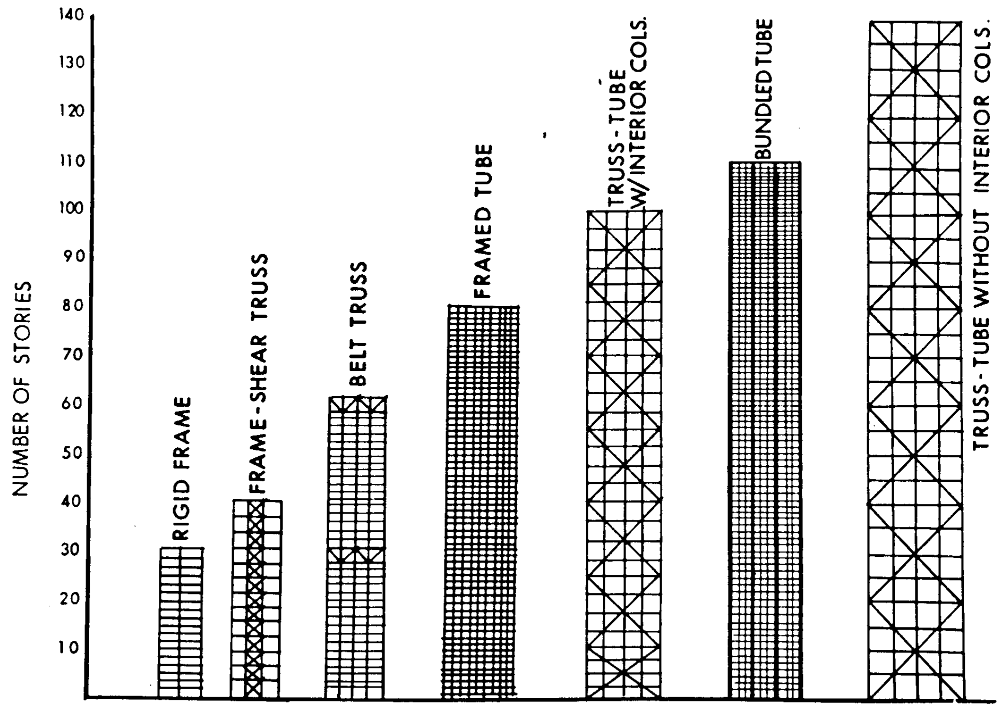

Fig. 2. Systems Chart

system has occupied the attention of tall building designers for nearly six decades until the early 1960s, with successive improvements in the same system without significant change in the form of the structure. However, during the decade of the 1960s, there was dramatic improvement in the technology of tall building structures. The system evolution had gone from the less efficient Vierendeel structural system to the more efficient cantilever systems represented by the Hancock Center, Sears Tower, and other tall buildings. It is interesting to note that a variety of systems in various combinations have been used since, but none exceed the efficiency of the cantilever.

In this sense, the cantilever concept represents the highest possible efficiency in a tall building system. The conventional Vierendeel steel-tiered system employed in many tall building structures, prior to the advent of cantilever systems, essentially consisted of beams and columns arranged in two directions to form a grid. Initially, connections between beams and columns were simple shear or semi-rigid moment connections at best. The developments in the early part of the twentieth century mainly consisted of enhancements in the quality of steel, steel shapes, and fasteners. Moment connections with increased rigidity were developed with extensive riveting and knee-bracing. Outstanding examples of these early tall building structures include the 790 ft. tall Woolworth Building in New York (ca. 1913) and the Empire State Building (ca. 1930). In these cases it also appears that heavy masonry claddings and partitions contributed as much to the lateral stiffness as the steel frame itself.

After World War II, a dramatically different architecture based on principles of Modernism emerged. Technological advances conforming to International Style architecture included larger open spaces with longer spans, a well-organized core and column grid, glazed curtain walls, sprayed on fire protection, and lighter partitions, to name a few. Vierendeel frame systems were still the primary choice, which involved bolted and welded rigid joints. As building heights increased beyond 30 stories, however, considerably more steel was required to develop adequate lateral stiffness. A rule-of-thumb at this time was to allow one pound of steel per sq. ft. per story. This amounts to 100 lbs. per sq. ft. for a 100-story building, which is clearly uneconomical in any context.

The idea that steel members can be assembled to form a three-dimensional resistive system began to emerge in the early 1960s. This was also coupled with the idea that different height ranges demand different system compositions to maintain a reasonable premium to resist lateral loads. The main proponent of this design trend was Fazlur Khan who systematically pursued a logical evolution of tall building systems that resulted in many innovative structural systems and recognizable built forms. Initially, Khan relation to building height (Khan, 1972; 1973). The least efficient system is the shear frame system that is economically suitable up to only 30 stories. This is followed by other structural systems culminating with an exterior diagonally-braced trussed-tube system as the most efficient. This development was extremely significant as it gave a logical platform against which the efficiency of newer systems can be measured. It also established a flexible design notion that system selection was the most important design event in the structural optimization process. This high-rise design chart has been extensively used and enhanced over time and is generally recognized as the greatest single contribution to high-rise building technology in the twentieth century. A systems chart proposed by Khan in 1972 is shown in Fig. 2 for steel buildings (Khan, 1972). He proposed a similar systems chart for concrete buildings.

It is in this context that the structural systems for the John Hancock Center and Sears Tower evolved. The realization of the systems approach to tall building design would not have been possible without the use of the computer as a design tool. Computer usage in practice was in its infancy in the early sixties and for the most part involved computerization of manual design calculations used in design offices. However, rapid development of analysis and design software for three-dimensional systems since then has helped to expedite the design and engineering processes.

The Program

The program for John Hancock Center includes multiple functions such as commercial space, parking, offices, apartments, and television transmission spaces yielding a total area of 2,800,000 sq. ft. The proposal to place all these functions in a single 100-story tower was based on the economies of possible structural systems. The single-tower concept was made possible as a result of the development of the trussed-tube at a unit structural steel quantity of 30 lbs. per sq. ft. The diagonalized form of the structural system had an equally radical impact on the architectural image of the building and, together with its revolutionary structure, constituted a daring leap by its designers – Bruce Graham and Fazlur Khan.

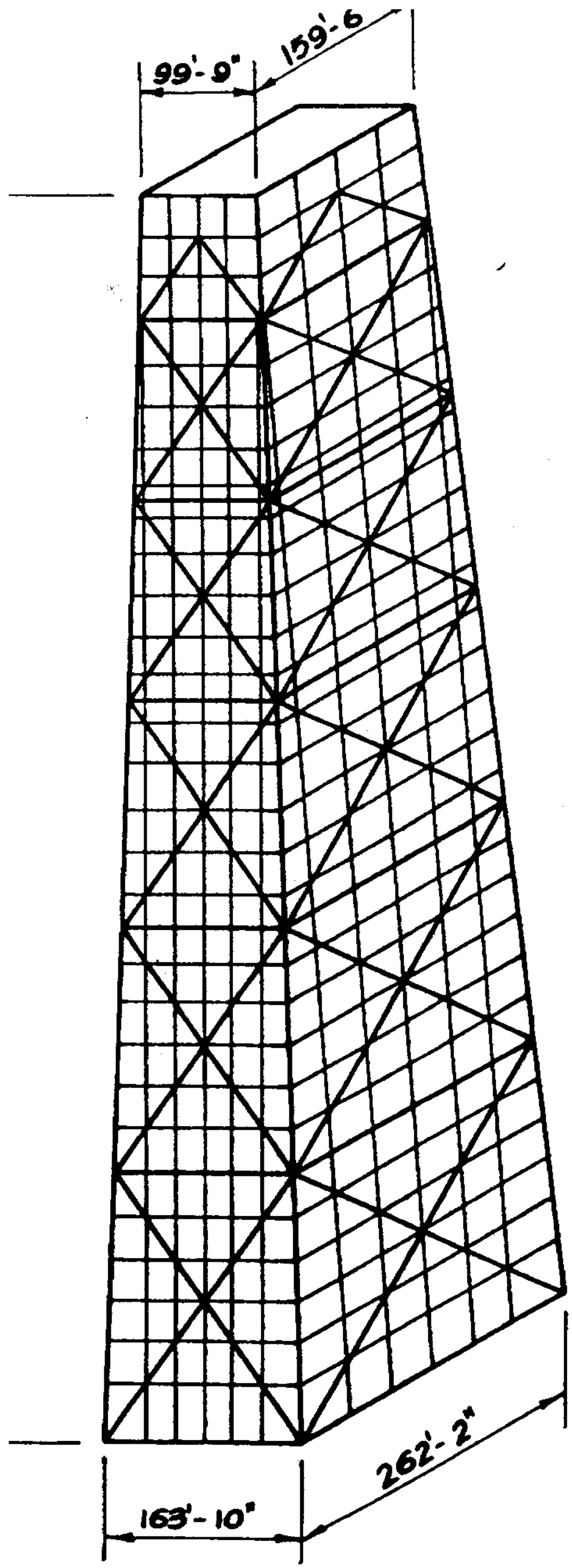

Fig. 3. Trussed-Tube Form.

The structure of the Hancock Center measures approximately 262 ft. x 164 ft. at the base and tapers to a topmost dimension of 160 ft. x 100 ft. at a height of about 1100 ft. above ground (Fig. 3). The commercial spaces and parking are placed at the bottom followed by 32 stories of offices, 50 stories of apartments, and spaces for television transmission, an observatory, and mechanical levels. The taper allows larger office floors with longer lease spans to be placed at the bottom and smaller apartment floor areas located toward the upper part. The overall impact of the tapered form allows for a continuous exterior structure. Additional benefits include reduced wind sail and improved shape-aspect with regard to wind dynamics.

The Concept and Behavior

The structural concept is based on an equivalent tubular system formed by a system of diagonal braces on the exterior, which ties all the exterior columns togetherand makes the whole system behave like a tapered rigid box (Fig. 3). The simplicity and straightforwardness of the structure and its impact on the architectural expression of the building can be readily observed. The essential character of the structure is created by the continuity of the exterior diagonal X-bracing on each face of the building. It is this three-dimensionality that makes it perform like a single tube structure in contrast to earlier systems that placed two-dimensional Vierendeel frames within the body of buildings in two directions. With the exterior trussed-tube assuming all the lateral load resistance, the interior gravity columns are required to carry only the gravity loads of floors in the most efficient manner. The absence of internal bracing or other internal resistive elements makes it possible to frame floors in a flexible manner for different functions, which is so essential for a multiple-use building.

Historically, diagonal bracing in truss form was introduced in bridge construction. The Eiffel Tower, built in 1860 in Paris, epitomizes the use of diagonals in towers. It also manifests the special characteristics of exposed steel construction:, aesthetic lightness and taut delineation. The four trussed-legs of the Eiffel Tower express the optimum relationship of exterior form and function to resist structural overturning. The Hancock Center embodies the same principle of utilizing the exterior form to resist overturning. However, unlike the relatively massive legs of the Eiffel Tower, its structure is organized to create the equivalent of a thin-walled exterior tube for maximum space efficiency and rigidity.

Other tall building pioneers were also interested in thin-walled tube structures. Myron Goldsmith investigated the evolution of exterior fascia diagonals and their architectural expression in a thesis in 1953while a student at the Illinois Institute of Technology (IIT) in Chicago. The diagonal bracing wasderived from diagrid type frames on each facade and by making the grid coarser, to a point of creating clear X shapes on each facade. Khan and Goldsmith also introduced this structural concept as a student thesis project at IIT in the early sixties, which involved a 60-story building outfitted with exterior diagonals. Simple manual calculations were performed together with a model load test to verify the lateral stiffness that indicated full box-like participation of fascia interior columns..

The possibilities of a diagonally braced structure are indicated in Fig. 4 – in which are shown a fine diagrid mesh in (a), the ultimate bracing with only corner columns in (b) and the tubular variety in (c). While the form noted in (b) is the most efficient, it poses certain practical fabrication problems relating to the size of the corner columns and diagonals. A distributed approach with some fascia interior columns as shown in (c) with a bay span of 30 ft. to 40 ft. is more practical and conventional. In order to channel loads into these columns secondary ties are necessary with the main tie restraining the horizontal spread of the X forms. Fig. 5 shows the load distribution characteristics of the fascia truss-frame and the effect of the ties (Khan, 1967). It is this type of load-flow that effectuates the behavior of an equivalent thin-walled tube. The geometric organization was followed rigorously in the Hancock Center structure, especially with respect to coordination of taper from each face with allocation of appropriate story heights for office and apartments.

The mega-truss form optimizes several parameters. The number of diagonals and, thereby, the window disruptions are minimized. Creating high 20-story tiers minimizes the number of column-diagonal connections. Normal column spacing of 40 ft. on the broad face and 25 ft. on the short face are used with conventional column and spandrel beam sizes. The diagonals act as inclined columns carrying their share of the gravity loads, which are always in compression even under extreme wind loads, thus simplifying the connection details using bearing type joints. 80% of the lateral sway under wind loads was determined from the cantilever component with the remainder from shear represented by axial deformation of the diagonals. Gravity forces generally controlled the system. The premium for wind loads was a factor of about 15% of the weight of the steel.

The Design

The design of the Hancock Center pushed the state-of-the-art in design at the time in many ways. The following is a brief retrospective discussion.

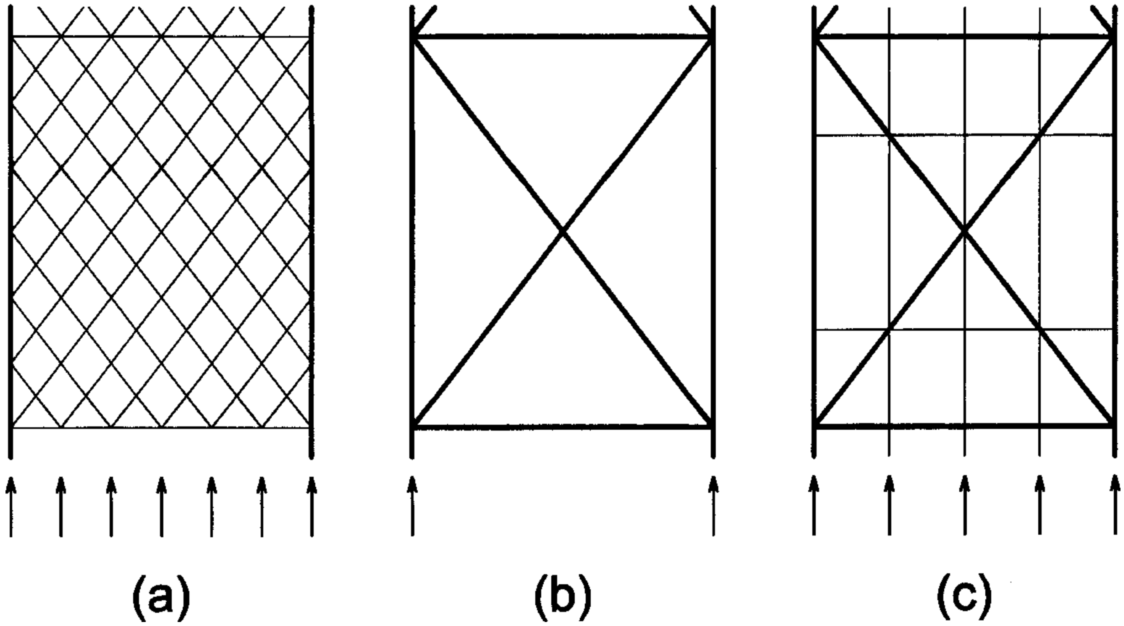

Fig. 4. Diagonal Possibilities

a. The Architecture. The architectural expression of the Hancock Center represents a radical departure from established aesthetics of the time. Initially critics labeled the building as too industrial, but over time it has come to symbolize the gutsy tradition of structural expression in Chicago. It is often characterized as super-rational, logical, and a representation of machine age architecture. It exhibits its true structure much in the tradition of bridge construction. Perhaps these are the qualities that are enduring over time.

The expressed diagonals of the façade have not adversely affected the quality of interior spaces. To the contrary, these elements are often coveted and decorated in a variety of ways, thus adding individual character to apartments. Furthermore, the multi-functional design fits very well with the commercial and residential character of the neighborhood. It has spawned a community of neighborhood tall buildings involving commercial / office and apartment mixes and, in this context, the collective mixed-use buildings constitute a unique neighborhood in the United States.

b. Computer-Aided Design. At the time that the Hancock Center was designed, computer-aided design in architectural and engineering practice was limited by the available memory capacity of hardware and the limitations of software.The largest structural problem that could be solved involved only 51 joints and three degrees of freedom and utilized an IBM 1620 computer (Kahn, et. al., 1966). However, larger capacity analysis programs such as Stress and other custom space-frame programs were being developed at a variety of academic institutions. In order to maximize the use of computers, preliminary analysis including behavioral studies on parts of structure, geometry determination, and data preparation was done on a small computer platform while the final analysis of the entire structure was performed at several academic facilities on large mainframe computers. This type of coordination may seem primitive in light of current capabilities; however, it allowed for maximum use of available computer capacities at the time.

c. Load Distribution Study. Load-flow analysis studies were performed on a three-tier fascia truss with two degrees of freedom for each joint as shown in Fig. 6. Such truss modules were analyzed at various locations in the building (Kahn, et. al., 1966). The aim of these parametric analyses was to determine the optimum proportions of diagonals and primary and secondary ties to produce a relatively uniform compressive stress in the columns, as would be the case of a bearing wall or a tube. The assertion was that if it worked over parts of the structure, it certainly would over the entire structure. Final analysis was not attempted until satisfactory results were obtained on the substructure analysis. Such substructure analysis gives valuable insight into structural behavior that is generally missed in full model analysis.

d. Wind Loads. Determination of statically applied wind loads required consultations with meteorologists and other wind consultants about the highest recorded wind events. It was determined that a factor of 1.25 would be applied to the Chicago building code in use at the time in order to calculate wind loads for normal allowable stress design. A factor of 1.4 was used to limit the steel stresses to 30 ksi. Statistical analysis of wind records was not available at the time; however, recent studies confirm these factors.

e. Wind Tunnel Studies. Wind tunnel studies on pressure models were performed in a steady-state wind tunnel as contrasted to today's boundary-layer wind tunnel. The main objective was to determine the shape-drag coefficients at various heights in the taper, all directed towards determination of the static wind loads.

f. Dynamic Behavior. Dynamic behavior was assessed from consideration of resonance with the vortex-shedding frequency. The fundamental sway periods were computed at 7.6 and 5.00 seconds with respect to weak and strong axes, utilizing a seven-mass model. Recent analysis involving masses at each floor indicates values of 7.05 and 4.9, and field-measured values of 6.8 and 4.76. These figures represent a good correlation. It was assumed that vortex-shedding frequencies would vary with height because of taper and consequently cannot organize to produce an effective dynamic force. This analysis, combined with a relatively stiffer system for sway and torsion, produced an acceptable dynamic behavior. The evaluation was more qualitative than quantitative, however. The measured values of damping at very low amplitudes was of the order of 0.6 percent of critical, perhaps indicating the predominance of axially transmitted forces.

g. Motion Perception. Occupant motion perception was assessed on the basis of pioneering experimental motion studies on human subjects at various attitudes to determine the threshold of perception and comparing it to analytical results obtained from forced vibration analysis of the building subjected to simulated wind gusts. Several time-variant wind gusts were applied after consultations with meteorologists. The gusts were simulated to correspond to the building period. The measured threshold of perception was in the range of 0.4 to 0.8 percent of g and the analysis predicted low down-wind accelerations. However, this methodology could not assess cross-wind accelerations, which generally predominate. They can only be determined with reasonable accuracy with force-balance or aero-elastic wind tunnel studies, which were beyond the state of the art at that time. This aspect of wind tunnel technology has undergone considerable improvement since the construction of the Hancock building. However, the structure

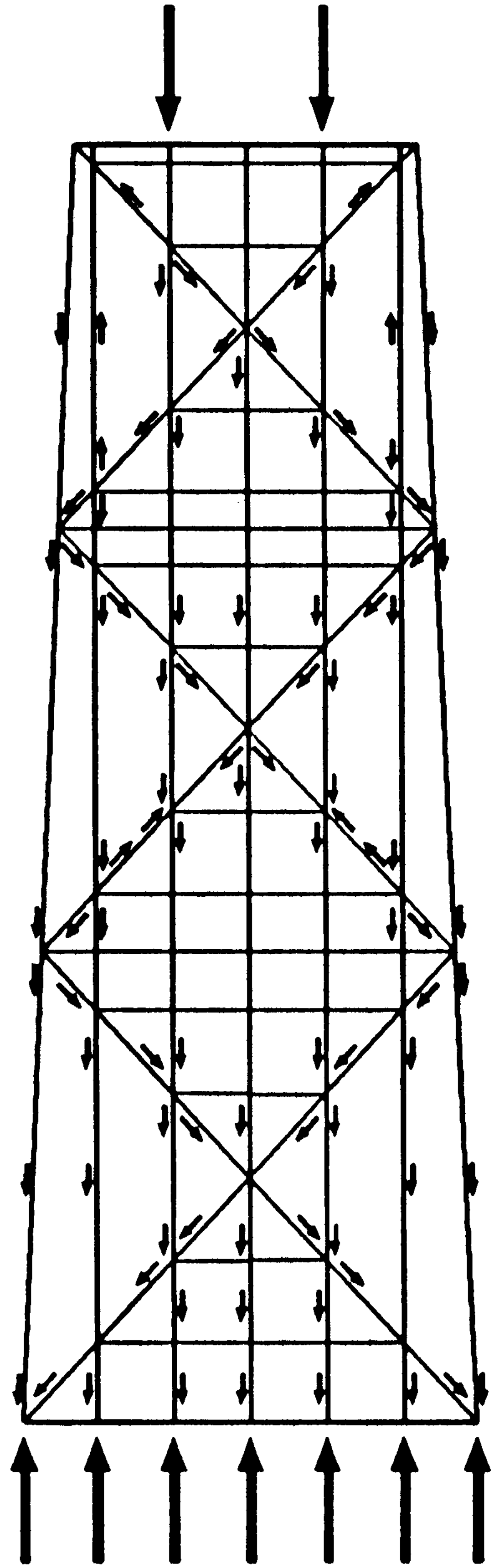

Fig. 5. Load Distribution

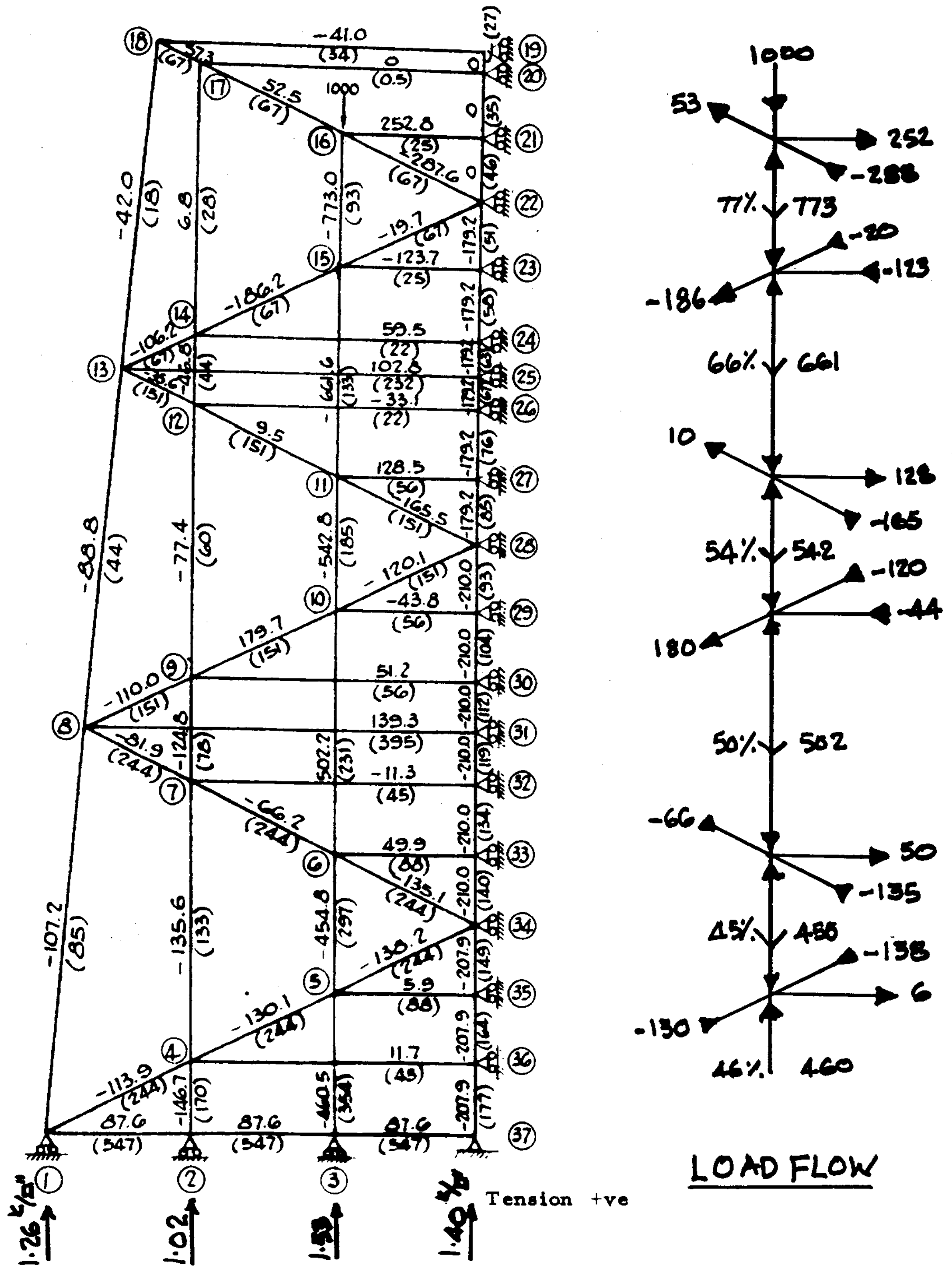

Fig. 6. Truss Module Load Flow

has performed well for wind loading over the last 30 years with no complaints on motion perception.

h. Floor Vibration. As floor-framing members became lighter because of non-encased fire protection and composite design, there was concern about user perceptibility of floor vibrations. Furthermore, the evaluation criteria were in only the formative stages at the time. Available research results in studies conducted in the early 1960s at the University of Kansas generally related the natural frequency of beams to amplitude caused by human occupation with the assumption of a certain amount of damping for different degrees of perceptibility. A program was undertaken to measure the natural frequencies of many similar existing floor beams and the perceptibility chart was used on a comparative basis with these measured values to assess the acceptability of vibration transmitted through the floor beams of the John Hancock Center. Even though the criteria have been refined over the years with new theoretical and experimental results, comparative evaluations with existing buildings that have performed well are still being used. A recent verification of floor beam vibration using current criteria indicates acceptable results.

i. Thermal Foreshortening. Since the primary system of columns and diagonals are placed at a relatively small distance beyond the glass line, there had been concern about the temperature differential between exterior and interior columns A maximum temperature differential of 5° F between the exterior and interior columns was sought, even with an extreme -10° F ambient temperature on the exterior. Cold-chamber experimental studies were undertaken to assure compliance with the criteria.

Joint Details

Primary system joint details among diagonals, columns, and ties involved the use of heavy gusset plates and weldments, especially at the corner joints. The member shapes (I-Section) and joint details were established with extensive discussions with steel suppliers and fabricators (i.e., American Bridge). This collaboration contributed to simplification and practicality in the concept and fabrication of the details. Generally, field welding was avoided between members and gusset-plated joints, but high strength bolted and butt-plated bearing joints were utilized. The gusset-plate assemblies were shop-fabricated. In each instance, the first corner-joint fabricated was measured for residual stresses due to welding. High levels of these stresses were noted and consequently all corner joint weldments were stress-relieved in gas-fired hot air ovens. Temperatures were raised to 1100° F in increments over time and gradually cooled to relieve the high stresses. Steel erection utilized exterior climbing-derrick cranes, a first in tall building construction. Similarly, various new welding techniques were introduced including vertical electroslag welding. The efficient and orderly coordination of steel fabrication and erection testifies to the successful collaboration between engineers and fabricators – a relationship that has practically disappeared in the current market.

Conclusions

The unique and optimum structural concept of the Hancock Center was made possible due to the collaborative effort among architects, engineers, and fabricators. If the structural system used for the building were to be designed and constructed according to current standards, few, if any, structural changes would be made. Exhaustive analysis conducted recently indicates no significant variations in its structural performance. However, advances in wind tunnel technology may require verification of dynamic behavior and motion perceptibility in accordance with current methodology. When such studies are made, the acceptable dynamic performance over the 30-year history of this tallest dwelling occupancy may lead to further calibration of criteria regarding motion perceptions.

The lessons learned from studying the design process of the John Hancock Center indicates that the introduction of any new building systems concept requires comprehensive examination of all relevant parameters and influences, even if they are only approximate. As demonstrated in the case of the Hancock Center, this approach will eventually lead to technological advances in tall building systems. This is true also for the Sears Tower that has a bundled-tube system with a skeletal frame expression. Meanwhile, the classic, structurally expressive architecture of tall buildings such as the Hancock Center and Sears Tower will continue to endure as symbols for architectural daring and the power of conceptualization.

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Reinforced Concrete Mega-Frame for Tall Residential Buildings

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Abstract

A new structural system (mega-frame) for tall reinforced concrete residential buildings is introduced. Globally speaking, the internal force distribution of the mega-frame is similar to that of the ordinary frame. However, the joint region of mega-frame is complex. To study the seismic behavior and failure mechanism of mega-frame joints, low cycle reversed loading tests of six joint specimens were conducted. It was found that flange walls could reduce the stresses in the joint region. Shear failure in mega girders and their connectors should be avoided. The behavior of joint regions is different from that of ordinary frame joints. Therefore, the design concept of the reinforced concrete mega-frame is not necessarily the same as that of ordinary frames. Three possible energy-dissipation mechanisms of the mega-frame structural system are discussed. Advantages and disadvantages of the failure of mega girders and joint regions are also discussed.

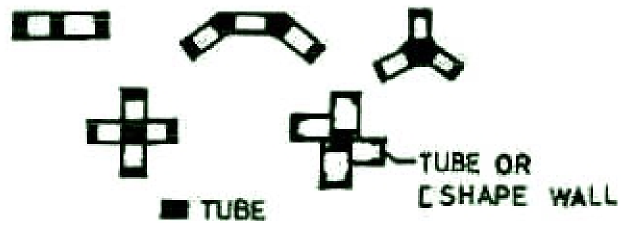

Fig. 1. Residential Building Plans

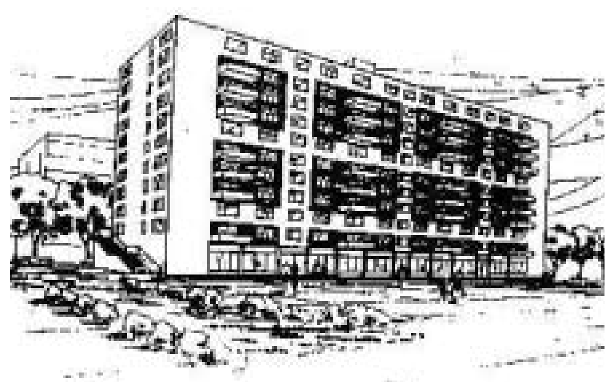

Fig. 2. Rectangluar Plan Residential Building

Introduction

Open space, safety, and reasonable cost are among the most important design considerations for tall residential buildings. This paper will demonstrate that amega-frame structure is an optimal system for these purposes. A mega-frame is composed of a series of main frames and several substructures. Each main frame consists of mega columns and mega girders. Mega columns are typically either the staircase cores or the elevator cores. Each one-story mega girder has a series of openings and is placed at three- or four- story intervals throughout the building. The main frame resists most of the horizontal and vertical Loads that are transferred from substructures. A substructure, which consists of three or four stories supported by a mega girder, mainly takes its own vertical loads.

The reinforced concrete mega-frame structure designed for tall residential buildings has various architectural plans, as shown in Fig.1 (Qingchang Hu, 1996). The perspective view of an 11-story residential building with the rectangular plan is shown in Fig.2 (Qingchang Hu, 1996). Mega-frame residential buildings have several advantages. First, a large open space for social intercourse can be planned in the story where mega girders are located because substructures are not necessarily continuous in the vertical direction. Second, the structural system and the materials for substructures can be different than those used in the main frame. This may result in savings in construction costs. Third, the mega-frame has a significant degree of lateral stiffness and can efficiently resist lateral loads. Fourth, the plan and space requirements for individual residential apartments depend on the construction of substructures, which may utilize standardized and prefabricated housing units.

The finite-element analysis results of a mega-frame show that the overall internal force distribution of a mega-frame is similar to that of an ordinary frame. However, the behavior of mega-frame components differs from that of ordinary beams, columns, and joints. For example, the mega girder is an I-shaped section with openings. The axial forces must exist in its top and bottom components, and the comparable shear forces exist in the connectors (walls between windows). Therefore, the failure mode of a mega girder is quite different from that of a conventional beam. Moreover, the joint of the mega-frame is a special condition, because it consists of thin walls and openings.

To study the behavior and the energy dissipation mechanism of a mega-frame, tests of six mega-frame joint specimens were carried out, and the theoretical analyses of mega-frame joints were performed. The main purposes of this paper are to synopsize the test results of mega-frame joints,. and to discuss the rational energy-dissipation mechanism and the design concepts of mega-frame systems under earthquake loads.

Tests of Joint Specimens

Test Objectives and Details of Specimens.

From a behavioral point of view, a mega-frame joint, including the joint region and its related mega girders and mega columns, is much more complex than the joint of an ordinary structural frame. Some seismic design concepts for moment-resisting frames, such as strong joint and strong column-weak beam, are not necessarily suitable for a mega-frame. To study these problems, low cycle reversed loading tests of six 1 / 4-scale mega-frame joint specimens were conducted. Fig.3 shows the joint specimen and the test setup.

Fig. 3. Joint Specimen and Test Setup

Typical mega-frame joint specimen and the denominations of each part are shown in Fig.4. Besides mega girders and mega columns, it is seen from the figure that the joint region of a mega-frame can be divided into nine parts, which could be classified into four types according to their position and performance under loads. They are identified as small joint zone, connecting beam, mega column pier along with flange wall, and kernel, respectively. There are four small joint zones, which connect the components of mega girders and mega columns. They look like conventional beam-column joints, however, their performance under loads is quite different from that of ordinary frame joints, as will be illustrated later. Connecting beams convert two single piers into coupled shear walls (a mega column), and contribute primarily to the stiffness of mega columns. Parts of mega column piers, which are located outside the joint region, are called flange walls. They strengthen the joint region and reduce the span of mega girders. The kernel, which may be perforated or solid, is the central part of the joint region. Due to the flanges of mega columns and mega girders, there must be a flange frame around the joint region. As illustrated above, the joint region of a mega-frame is complex and has many influencing factors that warrant extensive research.

Tests of mega-frame joints were conducted by Beijing Architectural Design and Research Institute (BADRI) and Tsinghua University in Beijing. Six specimens were tested in order to focus on the following aspects: a) the function of flange walls; b) the depth of connecting beams in the joint region; c) the width of connectors in mega girders; and, d) the nature and extent of reinforcement in the joint region. Details of test specimens are listed in Table 1.

Brief Introduction of the Test Results.

Failure mechanism of the six joint specimens can be divided into three types. First, shear failure happened both in connectors and in the bottom components of mega girders.

Specimens MJ-2 (Qingchang Hu et al., 1998), MFJ-1 and MFJ-2 (Xiaohui Chen et al., 1998) belong to this type. Second, flexural plastic hinges formed at the ends of mega girders where failure occurred. MFJ-3 and MFJ-4 (Guosong He et. al., 1999) are of this type. Third, connecting beams were destroyed by shear. Failure of MJ-1 (Qingchang Hu et. al., 1998) belongs to this type. The following conclusions can be drawn from these tests:

1. Flange walls have an important influence on the joint region. The difference between specimens MJ-1 and MJ-2 was that the latter was fabricated with flange walls and the former without. Flange walls contributed the most to the failure mode of MJ-2, which failed in the mega girders instead of the connecting beams. Flange walls reduced the stresses in these beams.

2. Shear failure could be avoided in the mega girders and their connectors. Shear failure is brittle, and it should

TABLE 1 DETAILS OF SPECIMENS

Series	*Flange walls*	Height of kernel opening (mm)	Depth of connectors (mm)	Concrete strength	Reinforcement details				Accomplishing Unit
					Top component of *mega girder*	Bottom component of *mega girder*	*Kernel*	Connectors	
MJ-1	No	375mm	380	C45	2Φ16(top) 2Φ16(bottom) ϕ5-40(stirrup)	2Φ16 2Φ16 ϕ5-50	2Φ16 2Φ16 ϕ4-50	4Φ16 ϕ5-30	BADRI
MJ-2	Yes	375mm	380	C30	2Φ16 2Φ16 ϕ5-30	Same as MJ-1	2Φ16 2Φ16 ϕ5-30	Same as MJ-1	BADRI
MFJ-1	Yes	No opening	380	$f_{cu} = 44MP_a$	2Φ12 2Φ12 ϕ5-50	2Φ12 2Φ14 ϕ5-70	ϕ5-50 (double layers)	4Φ14 ϕ5-50	Tsinghua University
MFJ-2	Yes	No opening	380	$f_{cu} = 35MP_a$	Same as MFJ-1	Same as MFJ-1	ϕ5-70 (single layer)	Same as MFJ-1	Tsinghua University
MFJ-3[1]	Yes	375mm	150	$f_{cu} = 42M$ $f_c = 35MP_a$	2ϕ10 2ϕ10 ϕ5-50	2Φ14 2Φ12 ϕ5-40	2Φ14+2Φ1 2 4Φ12 ϕ5-40	4Φ12 ϕ5-40	Tsinghua University
MFJ-4	Yes	550mm	150	$f_{cu} = 37M$ $f_c = 33.5MP_a$	Same as MFJ-3	Same as MFJ-3	2Φ12 2Φ12 ϕ5-40	Same as MFJ-3	Tsinghua University

1. 45° steel layers are arranged in the *joint region* of MFJ-3, which are ϕ5@35.4mm.
2. Φ : Grade II reinforcement; ϕ: Grade I reinforcement.

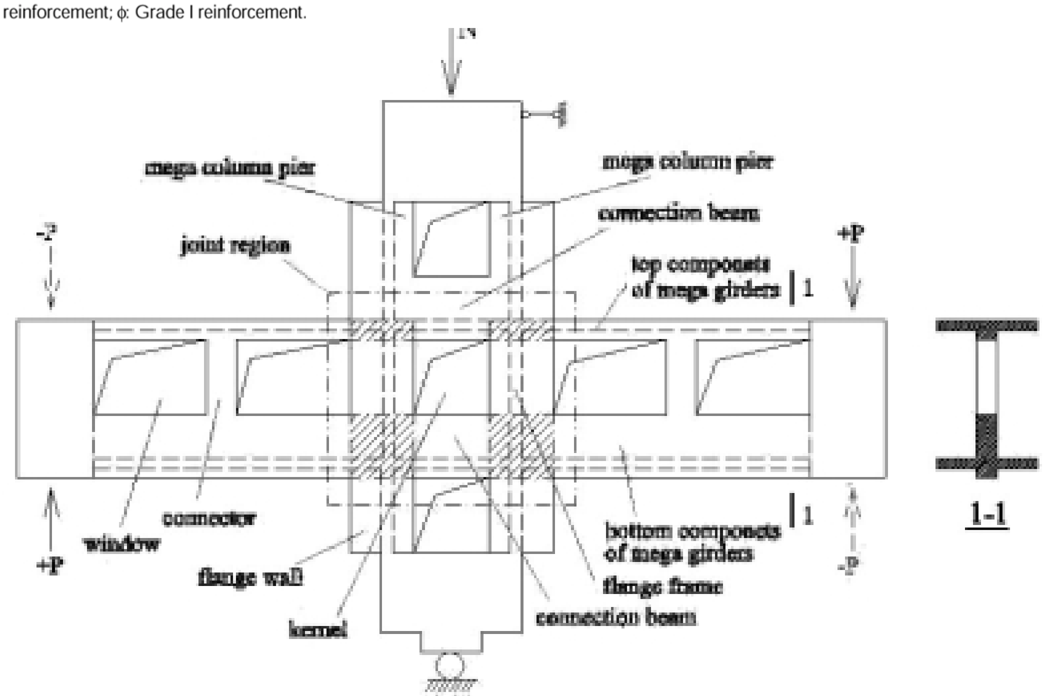

Fig. 4. Typical Mega-Frame Joint Denominations

be avoided. Both specimens MFJ-3 and MFJ-4 demonstrate that bending failures will occur in the mega girders and their connectors unless attention is given to designing them with the principle of strong shear-weak bending.

3. Connectors, which are the walls between windows in mega-frame systems, are the links of top and bottom components of mega girders. Shear failure of connectors, which should be avoided, reduces the integrity and the stiffness of mega girders. Adjusting the ratio of height to width of connectorsreduces the potential for shear failure. The ratio of height to width of connectors is 1.0 for MJ-1, MJ-2, MFJ-1 and MFJ-2, and is 2.5 for MFJ-3 and MFJ-4. The test results show that X-shaped cracks developed in connectors of the former, while bending cracks and only fine shear cracks appeared in the latter. The linear elastic finite-element analysis shows that changing the ratio from 1.0 to 2.5 has the effect of reducing axial force by about 23% in the bottom component of mega girders. This could also improve their overall performance.

4. Considerable stresses exist in the flange walls. As MFJ-4 shows, cracks appeared on the lower parts of the flange walls. A nonlinear finite element analysis demonstrates the corresponding results.

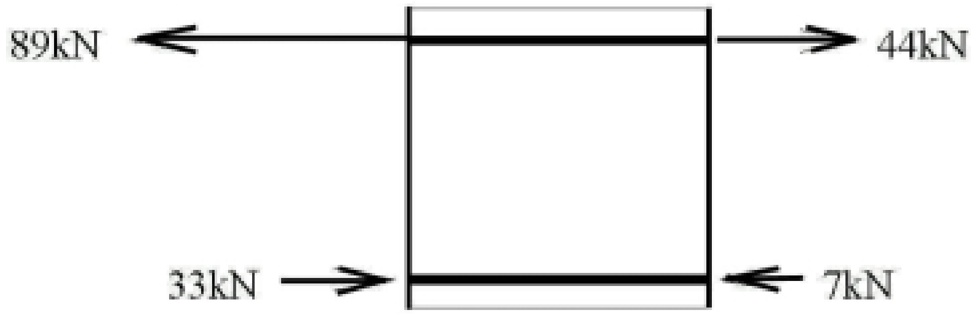

Fig. 5. Stress Distribution in Longitudinal Bars

5. The internal force distribution of small joint zones is different from that of ordinary frame joints. Fig. 5 illustrates the test result of the stress distribution in the longitudinal bars which cross the small joint zone. It can be seen that the bars withstand tension (or compression) at both ends simultaneously, while the longitudinal bars crossing the ordinary frame joint withstand opposing forces at the two ends (i.e., in tension and compression). It shows that the shear stresses and the bond stresses in small joint zones are much smaller than those in ordinary frame joints. Hence, the small joint zones are not prone to failure by shear and bond. No destruction in the small joint zones had been observed in the tests of all six joint specimens. The characteristics of small joint zones are beneficial for maintaining the integrity of the mega-frame itself.

Discussions on Energy Dissipation

The mega-frame structure for tall residential buildings is an innovative and efficient structural system. However, for seismic resistance, its energy-dissipation mechanism should be designed carefully. Some of the conclusions drawn from ordinary frames may not be suitable for mega-frame. The question as to what is the preferred energy-dissipation mechanism for mega-frame warrants discussion. The possible energy-dissipation mechanism of mega-frame systems can be classified into three categories depending upon the location where energy dissipation takes place: in mega girders, in mega columns, and in joint regions.
A few comments on the energy-dissipation mechanism of mega-frames under earthquake loading are as follows:

1. The test results show that it is not just shear failure alone that is of concern, but also the formation of plastic hinges followed by flexural failure in mega girders, that do not constitute satisfactory energy-dissipation mechanisms. Mega girders support several stories of substructures. They are the main elements for carrying vertical loads and for supporting substructures. Their large deformation produced by plastic hinges would damage substructures. The failure of hinged mega girders could lead to the collapse of substructures and the destruction of equipment such as pipelines. Repair work also would be harder after a moderate earthquake if mega girders were developed hinges.

2. Mega columns are the essential elements for supporting vertical loads. Hinging and failure of mega columns, especially in all piers, should be avoided since it would directly lead to the collapse of the whole structure.

3. Small joint zones are the essential links between mega columns and mega girders. Similar to ordinary joints, they should not be amenable to destruction.

4. Energy-dissipation elements in the joint region could be the connecting beams. Except for the small joint regions and mega column piers, yielding of the reinforcement in the connecting beams of the joint region is probably a better mechanism for absorbing earthquake energy. As is well known, these beams do not take any vertical loads, although they develop high shear and positive and negative bending moments at their two ends due to lateral loads. Therefore, their energy absorption capability is good if they could be designed as strong shear-weak bending elements.Furthermore, if these beams are destroyed, the coupling structures of mega columns revert back to the single walls. Although the single walls are slender, the structure would still be stable and the earthquake forces will be reduced because of decreased stiffness of the overall structural system.

5. If energy absorbers are installed in the kernel instead of an opening, energy dissipation is expected to occur there because it is subjected to shear deformation only.

Conclusions

Tests of six joint specimens have been conducted to study the behavior of a mega-frame system under cyclic loading and to study its rational energy-dissipation mechanism. Test results show that the force distribution at small joint zones is different from that at the ordinary frame joints. Therefore, bond failure of longitudinal bars and shear failure are not likely to occur there. This characteristic of small joint zones is beneficial to a mega-frame. Test results also show that shear failure, which is brittle, can be avoided by changing shapes of components and designing them carefully.

The possible energy-dissipation mechanism of a mega-frame system under earthquake loads has also been discussed. Clearly, a rational energy-dissipation mechanism should be allowed to develop in the connecting beams or in the energy absorbers installed in the kernel.The rational energy-dissipation mechanism and the failure mechanism constitute the basic concepts for the seismic design of mega-frames.

The primary goal of this paper is to present research on these concepts and to promote further study. Experiments of mega-frame joints are time-consuming and costly because the test specimens are large and complex. Therefore, it is not practical to study the behavior and all the influencing factors of mega-frame joints only by tests. A nonlinear finite element model and a computer program for studying mega-frame joints have been developed by the authors. They believe that they are more efficient tools for the parametric study of mega-frame joints. On the other hand, sophisticated nonlinear analyses, such as time-history analysis or push-over analysis, for the whole structure will be necessary to predict its behavior more precisely.

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The Reporter Calls

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It was the successful challenge by Petronas Towers (Malaysia) for "world's tallest building" title that opened a floodgate of telephone calls to the Council. Newspapers, magazines, television, radio, producers of programs – all have called. Besides responding to them, their questions have provided a stimulus to the Council leadership – questions that are shared here with you the reader.

Besides highlighting some of the most interesting questions posed – and our replies – the paper goes on to explain the impact of some of those questions, particularly those that led to the adoption by the Council of three additional height criteria, two of which are held by the Sears Tower and the other by the World Trade Center.

In the early days of the Council there weren't many reporters who called us. But after a year or so they learned about us and the calls grew in number, especially when we began to have overseas conferences. At those events in the 1970s there would be press conferences and occasional filming for the nightly TV news.

I'll never forget the New York Times reporter who visited Lehigh on the occasion of our first World Congress in 1972. He posed a question to Les Robertson (who would later become Chairman of the Council). "Would you go ahead with a project if the design was inappropriate?" (Even in the earliest days of the Council an important aspect of its work had to do with the environment and the impact of a tall building.) Les' reply: "Of course not." As we all know, that takes courage when the financial health of an office is at stake.

Later that reporter came to me. He had lots of questions because I was Chairman of the Council at the time. His last question: "Do you live in a tall building?" "No, but I work in one." Unfortunately, I didn't stop there, but made some other observations, not realizing how one of my comments might be mis-interpreted. "Oh, that's wonderful! I'd like to use that." I said, "No, that's off the record."

I saw the New York Times the next morning – the last day of the Congress. The reporter had written a good article. It looked like our Congress was getting good coverage. But then my eye reached the end. There was my "off the record" comment. I couldn't believe that he had ignored my stipulation. My colleagues greeted me that morning with an understandable lack of enthusiasm, and I could see why.

That experience taught me to answer questions accurately but not to volunteer anything that could be taken as a "negative". You can be sure that it will be used because reporters know what sells.

Then in March 1996 "disaster" struck Chicago. Petronas Towers, in far-away Kuala Lumpur, had taken away the "world's tallest" title from Sears. Weekly (and sometimes daily) calls then came to us from press and radio. It was as though flood gates had been opened.

During the previous year we had planned that our April Executive Committee meeting would be held in Chicago since it would coincide with a Congress of one of our Sponsoring Societies. As soon as that word got out, the calls came even more frequently. "Was the height criterion going to be re-examined at our meeting?" "Yes, it's been an item on our agenda for a year." CNN and NBC's "Today Show" arranged to interview me atop the Sears Tower in the days before the Executive meeting. It was the culmination of weeks of "media awareness" of the competition between Sears and Petronas. In the days leading up to the Executive Committee meeting, Council Headquarters received at least 75 phone calls from television, radio, newspaper, and magazine journalists - requesting information and interviews. Even "Spiderman" – the skyscraper-climbing daredevil – wanted a list of the 100 tallest buildings so he could climb them all, having already put Sears, World Trade Center, and Empire State under his belt.

Jay Leno joked on the Tonight Show with a line that all the Council did was once every ten years to look up in the sky and say, "Yep, that's the tallest!"

We held our Executive meeting – as scheduled and without the press in attendance. Blair Kamin, architecture critic for the Chicago Tribune, had been in touch with me earlier and I had mentioned to him that our meeting would finish about 4:00 p.m. and that he could stop by if he wished.

Four o'clock came, we opened the door and there was Blair – together with a host of others. Somehow the word had spread, and there was nothing for us to do but hold an impromptu press conference. There must have been a dozen TV cameras. Radio, newsmagazine, press, and news-service reporters came in to standing room only.

It was an “active” session, to say the least, with a number of reporters as emotionally involved as were the citizens of Chicago – with some strong reactions when they learned that the Council had simply re-affirmed a 60-year-old standard for measuring the height of a tall building (from sidewalk at the main entrance to the structural top). “But why don't the TV antennas count?” “Because flag poles, radio masts, and TV antennas have never been counted. And if they had been, Sears would not have held the record for 23 years because the antenna of New York's World Trade Center is higher (527 m, 1728 ft) than those of Sears (520 m, 1707 ft).”

Actually, on the plane en route to Chicago I’d been wondering what I could say that might be a palliative. And there were several Sears statistics that might be of help in that direction:

- It is world’s tallest steel building
- It is world's largest tall building: 418,000 sq m (4-1 / 2 million sq ft) of office space
- It has the highest occupied floor: 436 m (1431 ft)
- It has the highest roof top: 442 m (1450 ft)

When I was pointing out these features at the Executive meeting, Les Robertson suggested, “Why not establish three additional height categories in addition to structural or architectural top? These could be highest occupied floor, highest roof, and tip of any broadcast antenna.” This was agreed to and is now the Council's standard.

Shortly after those first calls started coming in at the time of the “Sears / Petronas saga”, I received letters from elementary school students in the Chicago area. One was written to Blair Kamin which he forwarded to me. It was written by Brett Wolff, a third grader in Eastville School. He said, in part, “…you just can’t take the title of ‘world's tallest building’ away from the Sears Tower and give it to the Petronas Twin Towers. It will upset the whole country. …Please count the Sears Tower antennae and give the title back to Sears Tower…not just for me, my class, Mrs. Elko, but for the rest of the country, including Chicago and the Sears Tower.”

Another letter was from Mrs. Meade’s third-grade class of the Moore School in Hinsdale, Illinois. Under the leadership of their Arts Awareness Coordinator, Linda McElherne, they had “studied’ the Sears Tower and the Petronas Tower drawings and concluded their deliberations with a vote. Quoting from their letter (which even included the ballot they’d used – complete with careful drawings). “The vote came back with 16 in favor of the flat-topped (Sears Tower) and 2 for the ornamental-topped (Petronas Towers).” It was signed by all 18 students.

After the Chicago Executive meeting was over, I had a morning to spare before my flight home to Bethlehem, so I visited Mrs. Meade’s class. I talked with the students about the Sears / Petronas issue, of skyscrapers, of engineering, and of civic pride. Duly impressed by their thoughtful discussion, I left the classroom that morning confident of our society's future if it's going to be in the hands of such as these. The children had given me much to think about, along with a number of wonderful letters and stories (one of which – by Jennifer Schwarzkopf – we featured as “page one” of our next newsletter). It was the culmination of a once-in-a-lifetime week in Chicago: Nationwide television. Front page headlines. Impassioned debates. Civic pride. Tradition. Innovation. And it was all topped off by the gatherings of friends and colleagues from around the world.

As Jennifer pointed out at the end of her “Tommy the Tower,” being the tallest may not be the most important thing after all. And this gets to the mission of the Council itself: to bring professionals together and to see that the latest knowledge from research laboratories and advanced design practice is collected and disseminated in such a way as to be useful to practitioners – who will create even better buildings to serve society.

Tall building developments began to proceed rapidly in Asia. Then there was the “Asian collapse” with local economics unable to justify the high-rise.

The questions started again, and so in the following I’ll attempt to identify some of those from among the hundreds that were posed by reporters.

Did Chicago really lose the title, home of the world’s tallest?

(This was the result of the Chicago excitement, of course.) It all depends on how you count. Initially there was a lot of resistance. But the Council’s adoption of four height criteria seems to have done the trick. In recent years there have been very few questions about the Petronas Towers being the tallest.

How high can you go?

Buildings have been designed to heights of a mile or more but of course not built. For the services engineer it's partly a matter of the elevators. We humans have ears that are sensitive to rapid changes in pressure. Above about 120 to 150 stories one could not use the kind of express elevators that whiz us to the top of the Sears Tower. (Rather we should say it’s the “going down” part.

That's when the pain sets in if the pressure increases is too fast.)

In the last analysis how high one can go depends on what the market can afford. It's the bottom line. If there are not enough tenants (or buyers in the case of condominiums) then the developer simply will not proceed. Of course, there are exceptions. In New York there's currently (2000) a building under construction that's called a "spec building" in which the developer has not waited for tenants to sign up. The economy has been so favorable that he's sure that providing the planned facilities at the right location will attract users. There's a similar case of a building under construction in Chicago.

How many tall buildings are there?

Our database, commenced in 1970 and still the most comprehensive, has about 10,000 buildings. My guess would be that there are at least 50,000 buildings ten or more stories high.

But compared to ordinary buildings, that's really not a very big number. Fly over such cities as Chicago and Los Angeles, and you'll see that the region of tall buildings is rather limited compared with the whole spread of the city – a relatively small number but a major impact.

How safe are you in a high-rise building?

I'd opt for the high-rise every time, especially in the case of earthquakes. Tall buildings are designed for them. You can't always be sure about the low-rise buildings, at least not until all of the buildings in the major cities are rehabilitated for earthquake protection.

Do tall buildings really move?

They certainly do. Mostly we don't see them move, but they move all the same. In an ordinary wind the top would move a matter of a few inches. But in the extreme design wind, buildings like the Empire State Building or the World Trade Center would move as much as three feet in each direction.

Isn't the high-rise a matter of ego?

There's no doubt that pride (a more appropriate term than "ego") is involved. But what's wrong with pride? Aren't we proud to have a baseball pennant-winner in our city? Of course we are.

Think of the country. In view of the fact that the developers were reasonably sure that the space could be let, the government of Malaysia certainly didn't stand in the way of the construction of the world's tallest towers. One surmises that it was encouraged.

hink of the city. The excitement that was generated in Chicago when Petronas passed Sears and became the world's tallest was really remarkable. I doubt that a press conference such as we had in Chicago on April 12, 1996 will ever be repeated. It was an amazing experience. Chicagoans were devastated, and ever since we keep reading of plans that could return the title to the windy city.

Think of the owner. If he can afford it and meet the standards of the building department and environmental limitations, wouldn't he be proud that his building was the tallest?

There's the design professional. I once attended a symposium on super-tall buildings, and someone said, "I'll bet every designer sitting around this table has a plan worked up for a world's tallest building – just in case a client comes along."

So there's nothing wrong with pride. What really counts is whether or not the market can support the project - and that all the appropriate environmental factors are considered.

So why are tall buildings needed?

Obviously it's more than urge. There's got to be a need. For office buildings it's a matter of agglomeration – getting many thousands of office workers together. When it comes to residences, it's a matter of energy conservation and land use. Suburbs require extensive land commitments, the cost of delivering urban services, and the energy consumed in transportation. Housing in the city conserves all of these.

One usually finds that tall buildings will be built in a particular city or country if three conditions apply: (1) the population of the country is growing, (2) cities and towns are growing, and (3) industrialization – and the service sector – are on the rise.

But since "everybody is on the computer" will we need tall buildings in the future?

Nothing in life is certain and one of the big unknowns is the impact of the computer and the Internet, telecommuting in particular. There is no doubt that working out of the home (or on the road) is on the rise. At the same time, however, one learns of the rapid increase in the need for office space in city centers by telecommuters and the E-industry.

In 1997 some 42% of North American companies had telecommuting arrangements, up from 33% in 1995. But only 7% of the employees took advantage of them. One out of five arrangements failed, for one of two reasons: “Employees have unrealistic expectations, and employers are afraid of losing control.” Common complaints: “I can't balance work and family.” “My dog barks too much.” “My house becomes a dropping off point for packages.” All the same, some 9- to 14-million American workers were telecommuters in 1999. Researchers agree on one thing: it works best in companies that have clear, formal, and tested policies.

A 1999 sign of the times: “Sun Microsystems recently signed a 10-year lease for 8290 sq m (89,152 sq ft of office space in New York’s World Trade Center.” Globe.com took 4410 sq m (47,000 sq ft at 120 Broadway. Platinum Technologies took a similar amount at 120 Broadway. These technology-based companies are moving into an area well known for high finance. More than 600 information technology organizations are located in the same area. Boosters are starting to call it “Silicon Alley.”

My personal opinion is that telecommuting will continue to grow, but not eliminate, the world-wide need for commercial office buildings. “Togetherness” counts. And, as we learned at the Council’s 5th World Congress, there are some cultures in which your office address simply can’t be virtual.

I've heard there’s a backlash against the high-rise?

The backlash is fairly well localized both in time and place. Some serious mistakes have been made in tall building planning. Pruitt Igoe in St. Louis is a classic example. That immediately created a wave of negativism, and there’s no denying that the public housing efforts that promoted the construction of dismal 10- to 15-story apartments all crowded together was a serious error. As everyone knows, those that were unsuccessful are coming down.

Rather than “backlash”, perhaps the question has to do with “resistance”. It’s not at all unusual for a neighborhood group to organize to protest the construction of a potential eyesore. If the case is good, they can succeed. On the other hand, a quality high-rise, in all respects, does not lead to a so-called backlash.

Is there any kind of a trend in tall building construction?

That’s as difficult to answer as are the questions that are put to economists about our financial future. To a certain extent, building a high-rise is always a gamble. When the economy of a region is vibrant then tall buildings are going to be built as long as the basic conditions are there. When things are not going along full-tilt it can be a big risk.

A trend we can underscore is that developers will continue to explore the opportunities of conversions, something that's happening a great deal as the loft buildings of major cities such as New York and Chicago are converted to apartments.

An evident trend is that most of the tallest buildings are going to be mixed-use buildings, combining office and residential facilities.

Even though there was an economic collapse in Asia we still hear about tall buildings in that part of the world.

As soon as the economy recovers then you’ll undoubtedly see more high-rise buildings in Asia. In countries like China, Thailand, and Indonesia it’s inevitable that when the economy improves, things will move because all of the right conditions are there: growing population, people moving to the city, and rapid industrialization.

Why do we keep trying to go taller?

As already noted, its basic justification lies in economics. And, to repeat, pride is going to be a factor – the pride of the country, the city, the owner, the designers, the builder. As we said, there’s nothing wrong with this. We take vicarious pleasure in winners with which we're connected: an Olympic games medalist, winner of the World Series, a basketball star.

Which is better steel or concrete?

Of course, I referred the reporter to the American Institute of Steel Construction and the American Concrete Institute. They have plenty of answers as to why their material is best. But as we know, it all depends. Each material does its job well. So comparison has to be made on the basis of other than material.

If one thinks of the “problems” of each material in a tall building. With steel it’s flexibility. The taller the building the more important it is to consider motion perception. That's the reason that in the tallest buildings there’s invariably a question during the design process as to the need for dampers.

For concrete (the ordinary strength variety) a height limit would be indirect, having to do with the space at ground level that has to be used for the columns. The greater the height, the less space is available for commercial use.

With the recent introduction of high-strength concrete, this is much less a factor, and motion perception can even become a required consideration for the tallest concrete buildings.

The tallest buildings now are almost invariably mixed construction, introduced in the 222 Riverside Plaza in Chicago by Fazlur Khan. As such a building goes up it looks like an ordinary steel building with its wide-flange columns and steel beams except that reinforced concrete quickly follows, surrounding the steel members. So mixed construction takes advantage of the benefits of rapid construction in steel and the economy of poured-in-place concrete.

In the case of earthquakes, the performance of the tallest buildings in San Francisco in the 1906 earthquake gave the impression that practically any steel building will stand up. (The damage to those buildings was not due to the earthquake, but to the fire that followed.) But then the Northridge earthquake came along (1994) and it was clear that, just as for concrete, something must always be done to assure continuity. Particular attention must be devoted to the connections to assure ductile action.

The labor market in the region has a lot to do with whether steel or concrete is used. In some cities the carpenters are so good at formwork that concrete could well be the choice. In other places there are steel erectors who are so experienced that those cities are known as "steel" towns. Another factor is the availability of manufacturing and fabricating facilities. Practically any country can develop manufacturing and labor facilities for concrete buildings. Steel, of course, requires manufacturing facilities and fabrication and erection force talent.

Are there any wooden skyscrapers?

Well, that gets into the question of what is a skyscraper. But if you mean something that has tallness as part of its characteristic then, yes. Personally I've seen seven-story timber tall buildings in Port Said, built in 1869 when the Suez Canal was under construction.

Some of the questions from reporters had to do with the origin of the Council and its mission. A few of them follow.

Why was the Council formed? How did it start?

At the 1968 IABSE Congress in New York, I was listening to Professor H. Beer give a summary of "research on tall steel buildings." I was struck with the thought that all of the research he described was completed but who was using the results? Professor Beer agreed that something should be done.

Then in February of 1969 in New Orleans at a meeting of the US Group of IABSE, I proposed a joint committee on tall buildings with the objective to prepare a monograph on the subject. Further, it would provide a focus for exchange and interaction between professionals. The group bought this idea immediately and the Joint Committee on Tall Buildings was on its way. Later, of course, it broadened its scope and became the Council on Tall Buildings and Urban Habitat. (In fact, the new designation was just catching up with a growing emphasis that began in the formative years. The first of what are now nearly 70 committees was called "Environment".

What is the Council's mission ? What does it do?

Its major function is to bring the wide variety of professionals together. Knowledge transfer is the mechanism for exchange. Apparently it's been successful if one can judge by the comment of Arthur Fox, Editor Emeritus of ENR, who said at a Lehigh meeting, "The Tall Building Council is the most successful technology transfer organization that I know of."

Of course, there were many many more questions, the above being just a sampling. And there were some that we couldn't answer – such as: How many people live or work in a tall building?

And maybe you have a question. Could it be about my comment to that New York Times reporter in 1972? Well, that's another story.

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