Shear Waves in Beams Subjected to Blast Loads

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Introduction

The behavior of structural members subjected to blast loads, e.g., in the case of boiler explosion or vehicle bomb attack, is an important concern in the assessment of structural safety in such adverse scenarios. For beams subjected to blast loads that are characterized by high magnitude pressure and short duration distributed along the length, it has been observed experimentally that failure is often initiated by direct shear failure at the supports and occurs shortly after load application, although diagonal shear failure along the beam is also observed sometimes. Such failure phenomenon can be explained based on Timoshenko beam theory where shear deformation and rotary inertia are accounted for, resulting in a model with a finite shear wave speed. Characteristic of wave propagation, the numerical solution demands fine mesh and small sampling time for accurate representation of waves that are responsible for initial built-up of stresses. As a result, the wave propagation problem is often solved numerically for an isolated beam or connections because the analysis of the whole structure is computationally prohibitive, if the wave characteristics are to be captured adequately. An understanding of the wave propagation phenomenon in building up the internal shear and moment during blast load events provides insights and opportunity for separating the wave propagation problem from the global analysis of the whole structure, thus permitting numerical analysis to be carried out with less computational efforts.

This work investigates the shear wave propagation in Timoshenko beams subjected to blast loads. Attempts are made to address the time scale over which shear wave propagation is important and where the global structural response can be adequately accounted for by global (coarse-meshed) structural dynamic analyses.

Timoshenko beam

The governing equations of Timoshenko beam under distributed load \( f(x,t) \) can be written as

\[
\begin{align*}
\gamma_a - c_1^2 \gamma_{aa} &= \frac{1}{\rho A} f_x - \Psi_{xx} \\
\Psi_a - c_1^2 \Psi_{aa} &= \left( \frac{c_1}{r} \right)^2 \gamma
\end{align*}
\]

where \( \Psi \) is the bending angle, \( \gamma \) is the shear angle, \( r = \sqrt{I / A} \) is the radius of gyration of the beam cross-section, \( \rho \) is the density of the medium, \( c_1 = \sqrt{G' / \rho} \) and \( c_2 = \sqrt{E / \rho} \) are the shear and longitudinal wave speed of the medium, respectively; \( G' \) is the effective shear modulus (absorbing the shear factor). It should be noted that, for a beam with only shear and no flexural deformation, i.e., \( \Psi(x,t) = 0 \), the governing equation in (1) reduces to the familiar form of a one-dimensional wave equation

\[
\gamma_a - c_1^2 \gamma_{aa} = \frac{1}{\rho A} f_x
\]

Thus, (1) and (2) can be viewed as a coupled system of wave equations, where the coupling terms are \( \Psi_a \) and \( (c_1 / r)^2 \gamma \) on the right hand side of (1) and (2), respectively. Note in particular that the response of \( \Psi \) in (2) is driven by \( \gamma \). One can thus view the system starting from rest as first excited by \( f \) through \( f_x \) in (1). This gives rise to response in \( \gamma \) that in turn excites response in \( \Psi \) according to (2), which is then “feedback” to affect \( \gamma \) through \( \Psi_{aa} \) in (1).

Wave response characteristics

Wave velocities

The Timoshenko beam admits two branches of dispersion relationship that characterize two bounded limits in the wave velocities as the wave number (defined as the reciprocal of wavelength) tends to infinity. They are the shear wave velocity \( c_1 = \sqrt{G' / \rho} \) and the longitudinal wave velocity \( c_2 = \sqrt{E / \rho} \). This is in contrast to the Bernoulli-Euler beam behavior, where the wave velocity is unbounded in the limit. The latter implies that impulsive disturbances can travel arbitrarily fast through the Bernoulli-Euler beam, which is physically impossible. This has significance in blast analysis of structures, as the applied load is mostly of large magnitude but short duration compared to the dynamic time-scale of the structure. It should be noted that the shear wave velocity \( c_1 \) is smaller than the longitudinal wave velocity \( c_2 \). Also, discontinuities in the shear response propagate at a speed of \( c_1 \).

Initial shear stress

When the beam is subjected to an ideal impulsive load of \( P \) (per unit length), it can be argued readily from (1) and (2) that the impulse imparts an initial velocity of \( \nu_a = P / \rho A \) uniformly along the beam. Also, the bending angle is negligibly small at the initial stage of wave propagation and the initial response is dominated by the shear component, that is, \( \gamma(x,t) \sim \gamma(x,t) \) for small \( t \). In view of these observations, the initial shear strain due to the wave initiated at the support may be obtained from consideration of a pure shear beam, i.e., with \( \Psi = 0 \). In particular, consider the deformation of the
beam near the support for small time $t$, as shown in Figure 1. At this instance, the shear wave initiated at the left support has traveled to the right for a distance of $c_1 t$, where $c_1 = \sqrt{G/\rho}$ is the shear wave speed. On the other hand, the unaffected portion of the beam has translated downward by a distance of $v_0 t$. Thus, the initial shear strain developed near the support is given by

$$\gamma_0 = \frac{v_0 t}{c_1 t} = \frac{v_0}{c_1} = \frac{P}{A \sqrt{G/\rho}}$$

(4)

The corresponding shear stress is given by

$$\tau_0 = G \gamma_0 = \frac{P}{A \sqrt{G/\rho}} \sqrt{G}$$

(5)

It should be noted that this initial stress is not directly related to conventional structural dynamic features, e.g., it is not related to the natural frequency of the beam. Instead, it is only related to the local material properties through which the wave propagates.

As an illustration, Figure 2 shows the normalized shear response of the left support of a 2.24m long simply supported steel beam subjected to a unit impulse of duration equal to $10^4$ of its natural period $T$ (=0.006s). The initial value of the normalized response is equal to unity, which validates (5). The spikes in the time history correspond to the instances where the shear discontinuities pass through the support, and are supported by $L/c_1$, i.e., the amount of time it takes for the shear wave to go from one support to another.

Figure 3 shows snapshots of shear distribution along the beam at different time instances. Note that the uniformly distributed blast load induces waves that start at the left and right supports. In the figure, the shear discontinuities that propagate from the left and right supports are marked by ‘L’ and ‘R’, with the arrow indicating the direction of propagation. Note that the shear discontinuity travels at a speed of $c_1$, while the true wave front travels ahead of the shear discontinuity at the longitudinal speed of $c_2$. The maximum shear response is affected by both the shear discontinuity and the response due to the true wave front. The magnitude of the shear discontinuity remains relatively unchanged as it propagates.

Maximum response characteristics with pulse duration

The maximum shear response is studied for a given impulse $P=q l_0$ but different load duration $t_d$ (and distributed load $q=P/l_d$) to investigate the time scale over which wave phenomenon is relevant. The beam is assumed to be a rectangular steel section with $E=205$GPa, $G=78.85$GPa, $A=1$ and $t_d=0.01$. The shear response is normalized by $\tau_0$, and the maximum response is observed at $t_d=0.0175$ with $P=1.0$ and $q=0.01$. The results are shown in Figure 4, where the maximum shear response is plotted as a function of $t_d$, with the dotted line indicating the natural period $T$ of the beam. The figure shows that the maximum response is significantly affected by the load duration $t_d$, with the maximum response occurring at $t_d=0.0175$.

Figure 1. Shear strain at simply support

Figure 2. Normalized shear stress history

(a) $t/T = 0.0083$

(b) $t/T = 0.05$

(c) $t/T = 0.1$

Figure 3. Shear distribution at different time instances
The equivalent shear factor $k$ is taken as 5/6. The shear wave speed is calculated to be $c_\tau = 2.87 \text{km/s}$ and the longitudinal wave speed is $c_\lambda = 5.11 \text{km/s}$. The response is computed using finite element method. Classic two-node cubic elements with no shear-locking effect and consistent mass matrix are used in finite element model. A large number of elements are required to capture the exact details of deformation that are characterized by wave propagation. Twelve hundred beam elements are used in this study, which is found to give reasonable convergence of results. Newmark average time stepping algorithm is adopted using time intervals ranging from $1.0 \times 10^{-9}$ to $5.0 \times 10^{-7}$ for different load durations. The fundamental period is obtained numerically as $T = 0.006 \text{ s}$, which corresponds to a single curvature bending mode.

Figure 4 shows the maximum shear stress (over the whole time history of response) at the support versus the load duration $t_d$. The corresponding results obtained by using an Euler beam are shown for comparison.

The results in the shear stress are normalized by the initial shear stress $\tau_0$ given by (5). As shown in Figure 4, for the Timoshenko beam, as the load duration decreases, the maximum normalized shear stress converges. Note that the results of the Bernoulli-Euler beam tend to infinity as the load duration decreases. This difference is a consequence of the boundedness of the wave velocity as the wave number increases. In the case of a Timoshenko beam, as $t_d \to 0$, this involves only accelerating a portion of the mass of the beam through which the wave has propagated, which is of the order of $t_d$. As a result, the force required to realize this acceleration, equal to the product of mass (of the order of $t_d$) affected and acceleration (of the order of $v_0/t_d$), is finite. On the contrary, in the case of an Euler beam, involving the left end of the beam to accelerate from 0 to a velocity of $v_0$ involves accelerating the whole mass of the beam, since the whole beam can be set into motion in no time, as a consequence of the infinite wave speed. As a result, the force required to realize this acceleration is infinite, since the mass affected is not of infinitesimal order $t_d$.

As the load duration increases, the results asymptotically tend to zero for both the Timoshenko beam and Bernoulli-Euler beam. This is because in this ‘quasi-static’ regime, the maximum shear force is asymptotically equal to its dynamic step response (i.e., twice of the static value), i.e., $2 \times (qL/2) = PL/t_d$, which tends to zero as $t_d \to \infty$. The regime between $t_d / T = 0.001$ and $t_d / T = 1.0$ can be considered as structural dynamic dominant. Finally, the results of both the Timoshenko beam and the Bernoulli-Euler beam coincide for around $t_d / T \geq 0.1$. This is the regime where shear deformation and rotary inertia can be neglected and conventional structural dynamic notions are applicable.

**Conclusions**

The wave propagation phenomenon in building up the internal shear in the Timoshenko beam subjected to blast loads has been the focus of this work. The study shows that the maximum shear response due to impulse loads is related to local wave propagation rather than global dynamics, and it occurs shortly after load initiation. Current research examines the implications of this to structural response to blast events and post-failure nonlinear behavior that is relevant.
Reliability and Failure Analysis of Complex Systems

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Introduction

The reliability of a system addresses the question of ‘how likely failure occurs,’ and failure analysis is concerned with ‘what happens when failure occurs.’ The issues evolved around reliability and failure analysis are important for performance assessment and for design provision, since engineering systems should be designed to have low failure probabilities and the probable scenario that may occur in case of failure provides vital information for failure prevention and loss mitigation, especially in catastrophic failures.

Within a probabilistic framework, failure analysis requires the assessment of response attributes conditional on the failure event and quantities formulated as a conditional expectation are frequently encountered. For example, the expected first-passage time over a given serviceability or ultimate limit state gives an idea of how long the system can sustain before failure occurs and provides guidance on renewal cycles. In a system with \( m \) components connected in series, the most vulnerable component can be identified as the one with the highest value of conditional probability \( P(F_i|F) \), where \( F_i \) denotes the failure of the \( i \)-th component and \( F = \bigcup_{i=1}^{m} F_i \) is the system failure of any one of the components; in a structural system, the conditional collapse probability given that damage of some load bearing components (e.g., columns) has occurred has been advocated by some researchers as a measure of structural redundancy. The conditional distribution of the uncertain variables also gives an idea of the probable cause of failure. When compared to the unconditional distribution, the conditional distribution of a set of random variables \( \theta \) reflects how important the corresponding uncertain parameters are in affecting failure. In fact, it can be readily argued using Bayes’ Theorem that \( P(F|\theta) \) is insensitive to the set of random variables \( \theta \) if the conditional distribution of \( \theta \) varies in a similar fashion as the unconditional one, and vice versa.

Reliability and failure analysis problems are ‘NP’ in computer science terminology, in the sense that there is no universal algorithm for its general solution in polynomial time as the complexity of the problem increases. Complexity can be due to state-space dimension of a system, nonlinearity, bifurcation behaviour, etc. A major source of complexity is that in relevant in probabilistic analysis is the dimension of the problem, namely, the number of random variables. Unfortunately, this is often quite large (and theoretically infinite) in many engineering applications. For example, a stochastic description of an excitation time history (e.g., earthquake motion, ocean wave) theoretically requires an infinite number of random variables (in order to model the whole frequency bandwidth) and can easily require thousands depending on the sampling frequency and duration of study. The random field description of soil properties on a spatial grid is another example. This complexity requires reliability methods to be efficient yet robust to the dimension of the problem. Efficiency and robustness, however, are often competing factors of an algorithm where a trade-off should be played.

From a simulation point of view, reliability and failure analysis can be performed by investigating the statistics of system behaviour corresponding to random samples that are distributed as the conditional distribution. This necessitates efficient simulation of ‘conditional samples,’ which belongs to the problem of simulating random samples according to an implicitly known distribution, and is a highly non-trivial task. Standard Monte Carlo simulation is well-known to be the most robust procedure regardless of problem complexity, but it is not efficient when the failure probability is small, which is commonly encountered in engineering applications. Similarly, the method can be used for generating the conditional samples, but its efficiency aspect is even worse. Essentially, it requires on average \( 1/P(F) \) trials (and hence system analyses) to obtain one conditional sample. As a rule of thumb, to achieve a failure probability estimate within a standard error of 30%, it requires on average \( 10/P(F) \) samples, or equivalently, 10 ‘failed samples.’ Nevertheless, in failure analysis, 10 conditional samples are usually not enough for making an accurate prediction by statistical averaging. This means that the required computational effort is even more demanding than that for reliability analysis. In particular, suppose \( P(F)=0.001 \) and 100 conditional samples are required for failure analysis, then it requires on average \( 100/0.001=100,000 \) system analyses, which is computationally prohibitive. These observations call for more efficient methods for the problem. In wake of Moore’s law that predicts a long-run exponential growth in CPU speed, however, the robustness of a reliability method assumes a vital importance since any gain in efficiency at the expense of robustness will soon become unworthy.

Subset Simulation

A method called Subset Simulation [1] has been developed as an adaptive simulation procedure for efficient yet robust solution for reliability and failure analysis of engineering systems. It stems from the idea that a small failure probability can be expressed as a product of larger conditional failure probabilities of some intermediate failure events. The problem of rare event simulation is thus converted into a series of more frequent event simulation problems. It also makes use of Markov Chain Monte Carlo simulation (MCMC) to efficiently generate conditional samples. MCMC is a powerful simulation approach that has wide spread applications in statistical physics, Bayesian statistics, image processing, econometrics, biostatistics, phylogeny, etc [2]. During Subset Simulation, the conditional samples are generated from a specially-designed Markov chain whose limiting stationary distribution is the target conditional distribution. The samples populate gradually from the frequently occurring region towards the rare failure region of decreasing failure probability. The procedure is illustrated in Figure 1.
Figure 1. Illustration of Subset Simulation procedure

(a) Level 0 (initial phase): Monte Carlo simulation
(b) Level 0: Adaptive selection of first intermediate threshold level
(c) Level 1: Markov chain Monte Carlo simulation
(d) Level 1: Adaptive selection of second intermediate threshold level

Illustrative Applications

Figure 2(a) shows the failure probability versus peak inter-storey drift ratio of a 6-storey-3-bay moment resisting steel frame subjected to stochastic earthquake excitation modelled as a point source according to the work of Atkinson and Silva. The scenario corresponds to an event of (moment) magnitude 7 at a nominal distance of 20km from the building. The results are computed using 1,500 samples where the standard error of the estimate at the failure probability level of 0.001 is about 30%. This should be contrasted with 10,000 samples if standard Monte Carlo were used. Figure 2(b) shows

the spectrum of the white noise sequences that generates the earthquake conditional on failure of increasing inter-storey drift levels. As the failure level increases, the conditional spectrum develops a peak at around 0.5Hz, which corresponds to the natural frequency of the structure. This indicates that when the shaking intensity is fixed, resonance effect is the probable cause of failure. These results are obtained based on the 1,500 samples that are used in Figure 2(a). Note that 50,000 samples are required to produce the results in Figure 2(b) if standard Monte Carlo were adopted.

Figure 3 shows the results when the magnitude and location of the earthquake event are not fixed but are considered to be uncertain, which addresses the seismic risk problem. In this case the moment magnitude is assumed to follow a truncated exponential distribution according to Gutenberg and Richter, and the location is assumed to be likely within a circular region of 50km around the building. A power law decay trend is observed in Figure 3(a), a common trend in the study of natural hazards. Unlike Figure 2(b), there is no significant spectrum peak in Figure 3(b), suggesting that resonance effect is not relevant in this case. Rather, the migration of the conditional samples (plotted on the magnitude-distance space) in Figure 4 towards the large-magnitude-small-distance region indicates that failure is governed by shaking intensity.

(a) Level 0              (b) Level 1               (c) Level 2

Figure 4. Conditional distribution of earthquake magnitude and epicentral distance

Conclusions

Simulation provides a versatile tool for reliability and failure analysis. The latter necessitates efficient generation of conditional samples. Standard Monte Carlo simulation is most robust to the complexity of the problem but it is computationally inefficient when dealing with rare failure scenarios. Subset Simulation has been developed for efficient yet robust reliability and failure analysis. Future research directions point towards improving efficiency of the algorithm to different class of problems, in view of the trade-off between efficiency and robustness.

References


Valuation Techniques for Infrastructure Investment Decisions – Beyond NPV

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Introduction

The requirements to expand, modernise and sustain infrastructure facilities amidst dynamic changes and capital constraints worldwide are well known, and these conditions have forced public infrastructure owners to adopt innovative methods to provide facilities and services necessary to support economic productivity and social welfare. Quadrant II projects (Miller, 2000), such as Design-Build-Operate-Finance, have been resurrected as one viable strategy to deliver needed capital facilities. In these projects, it is incumbent upon the owner to properly evaluate a project’s potential for non-recourse project financing before requesting proposals from private vendors. Evaluating the economic viability of such projects from the perspective of potential investors forces a public owner to independently consider the market risk.

Economic feasibility analysis has not traditionally been a skill widely harboured by public leaders or engineering professionals. In addition, many infrastructure investments possess option-like features such as deferment or staged investment that traditional valuation methods cannot represent, so quantifying the value of such options can be quite significant to the timing of investments. Given these circumstances, the intent of this article is to give a primer on real option valuation in infrastructure projects. The first portion of the article provides an overview of valuation methods. The second portion illustrates two case applications: the first has been completed for a toll road project in the United States; the second is currently under research for a power plant project in India.

Traditional valuation and its limitations

The use of discounted cash flows (DCF) to determine net present value (NPV) is one of the most common methods for establishing the value of a project (Grinblatt and Titman, 1998). This methodology discounts future expected cash flows at a pre-determined discount rate. Recent literatures have elaborated on the shortcomings of this “naïve” evaluation strategy:

(i) Simple NPV method fails to capture managerial flexibility in decision-making; hence NPV often underestimates the true value of a project by not taking into account the various strategic options that a manager has. Taking an expected value of uncertain cash flows ignores these options, as managers need not commit to the downside risk of a project at the later stages of investment.

(ii) An asset’s economic viability determined by NPV analysis is most sensitive to the discount rate. Despite this sensitivity, discount rate selection remains elusive even for seasoned financial analysts.

Real option models

Traditional valuation only works well for selected engineering investments, such as equipment replacement, where the main benefit is cost reduction. Investments in general, however, often create future growth opportunities (e.g., follow on development if product demand is favorable) or they have contingency possibilities (e.g., delay or abandon project). In effect, the risk of subsequent cash flows can change as development proceeds or new information is received. In such cases, DCF methods understate the value of this flexibility. Real option models as an alternative valuation technique has been proposed and developed to capture these values of flexibility. In general, real option models fall into two categories: (a) continuous-time models and (b) discrete-time models.

Continuous-time models represent one or more variables as stochastic processes. For infrastructure projects, analysts often select the value of the underlying facility, which is the present value of cash flows derived from the completed project or specific operating assets, as the variable of interest. In finance, a common stochastic process used to model the underlying asset value is the geometric Brownian motion:

\[ dV = \mu V dt + \sigma V dz \]

whereby \( dz \) represents the basic Wiener process:

\[ dz = \epsilon \sqrt{dt} \]

and \( \epsilon \) being a random variable with a standardized normal distribution \( \phi (0,1) \).

Common forms of discrete-time models include the binomial model, the trinomial model and the lattice model. One class of models essentially represents “economically corrected” versions of decision-tree analysis so that the problems of payoff structure, risk characteristics and non-constant discount rates – all of which are primarily due to the flexibility and asymmetry embedded in the decision-making process – can be overcome. This is done by converting the real world situation into a risk-neutral scenario by virtue of the Girsanov’s theorem (Mikosch, 2000).

Case application 1: Dulles Greenway in northern Virginia, U.S.

The project’s economic viability is evaluated by using both traditional and option valuation techniques. The project
background, cash flow models and other details can be found in Wooldridge et al. (2002). In the traditional evaluation, a discount rate of 15.6% was derived. A base case analysis follows the consortium’s original expectation of an average of 20,000 vehicles per day in the first year of operation; all other parameters are set as described previously. Under these conditions, the project’s NPV is negative $86.3 million as illustrated in Figure 1, so the investment appears rather suspicious. Both one-way and two-way sensitivity analysis are also conducted. Traditional valuation suggests that the project is not exceedingly robust, and this leaves a decision-maker in a quandary. The investment decision hinges upon the strength of the traffic forecasts and the confidence in the judgment about the linkage between the project and general economic conditions. The project’s developers, however, have another potential alternative; they could defer the project. Deferment could allow the acquisition of better information and the observation of economic growth in the outlying regions.

A simple binomial model is used to value the option of deferring. Figure 2 illustrates the model’s basic set-up in a risk-neutral world, which is used to establish the risk-neutral probabilities for future states of demand. These risk-neutral probabilities can then be used to evaluate the value of the project embedded with the deferral option as illustrated in Figure 3. It is determined that NPV_U = $56.0 million at a volume of 34,000 vehicles – the project is attractive and development proceeds; NPV_L = $0 at a volume of 10,000 vehicles – the project is unattractive and development does not commence. The value of the project with the embedded deferral option is thus calculated as $25.5 million. The value of the deferment option by itself is huge: $111.8 million ($25.5M - ($86.3M)); ignoring this value will grossly underestimate the potential value of the project. These results are a far cry from the base case NPV of negative $86.3 million.

Case application 2: Enron’s Dabhol Power Project in India

The Dabhol power project was initiated in 1992 in the Indian state of Maharashtra. The project has a Build-Own-Operate (BOO) setting and the power purchase agreement was signed between the Maharashtra State Electricity Board (MSEB)
Conclusions

Project financing arrangements often encompass various flexibilities and risk mitigation measures. The techniques presented illustrate the limitations of valuing projects traditionally and the supremacy of deploying a binomial model for valuing the option to defer. This binomial model represents only one of the many ways of applying the real option techniques. This option model can be characterized as an “economically corrected” decision-tree. The real option techniques can aid in the analysis of a project’s private finance potential, investment timing and thus facilitate better decision making during negotiation and procurement.

References


Strategic Analysis of the Chinese Construction Industry

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Introduction

Singapore’s establishment as an industrialised country, coupled with limited land resources, has seen its domestic construction market becoming more matured and saturated. In no time, local engineering and construction (E&C) firms have to venture abroad. With a GDP growth of more than 7% in the past few years and at least in the near future, China stands as an attractive market to establish a “second base” for Singapore firms. Although facing lower language barrier and shorter “psychic” distance (cultural factors etc.) as compared to their Western counterparts, many entrants from Singapore still experience losses in their initial encounter with the market and operating environment in China.

In the long term, the research initiative presented here aims to demystify some of these aspects. The complexity of the entire picture nonetheless warrants breaking it up into smaller stages. The current stage focuses on understanding how local construction companies in China are coping with their environment – even many of them are plagued by poor profits, with some on the brink of bankruptcy. Ultimately, the following question is of interest: If the locals are suffering themselves, would Singapore E&C firms stand a chance to survive there?

The role of strategic management theories

Although numerous publications exist to report strategic
Issues faced by construction firms in China, many of them are merely presenting opinions, with a general lack of theoretical content. In many ways, these diverse opinions have to be streamlined so that a more coherent understanding of strategic issues faced by these construction firms can be achieved. To achieve this, strategic management theories and experience developed in the West for the past forty years should be helpful. As a matter of fact, strategy is hard. Its intellectual foundations are drawn from various primary disciplines including finance and economics, organizational sociology, political science, and cognitive psychology, all of which are constantly evolving at their own pace (Rumelt et al., 1994). For example, the development of game theory not only expands the frontier of economics, but also contributes to strategic management through its adoption in structuring competitive strategy for oligopolies. In addition, the actors and audience of strategy are diverse, ranging from academicians to managers and consultants concentrating on different industries and sectors. Naturally, this heterogeneous composition of disciplines and players ensures that strategic management theories are diverse.

Whittington’s (2001) work is found to be extremely helpful in streamlining such diversity and narrowing down to four basic conceptions of strategy – rational, fatalistic, pragmatic, and relativist (Figure 1). He finds that these distinct schools of thoughts can essentially be mapped along two axes: outcomes of strategy and the processes by which it is made. The vertical axis examines the degree of variation of strategic intent and outcomes produced. This may represent profit maximization per se at one extreme, or accommodation for other complex priorities such as social responsibilities at the other end of the spectrum. The horizontal axis considers the fact of whether such outcomes are derived from deliberate planning, calculation and formulation, or simply as an emerging product of accidents, chance, and social and organizational inertia. The radically different implications on strategy are hence read off from the relative positions along the two axes in the diagram.

Each of four approaches listed in Figure 1 has its own strengths, weaknesses, and specific implementation aspects (Cheah, 2002). Most stable market environments, such as in the U.S. and Japan, mainly require adoption of one of the approaches (e.g. Classical for U.S.; Systemic for Japan).

Analysis of the environment of Chinese construction industry: the top-down method

The top-down method of industry analysis (Benninga, 1997) is used for analysing the environment of the Chinese construction industry, as illustrated in Figure 2. It is found that analysis of the Chinese environment requires a mix of the four generic approaches to deal with different aspects of the strategic sphere. For example, analysis of the external, larger environment would make use of institutional theories belonging to the Systemic branch while analysis of industrial environment at the micro-level would utilise Classical theories. Finally, development of sustainable distinctive competencies in the turbulent Chinese market environment would rely on the Processual school of thought.

Selected analytical results

Overall, China’s macroeconomic and business environment are not highly regarded. An example of this is the findings published by Political & Economic Risk Consultancy, shown in Figure 3. Essential elements and risk variables summing up to this aggregate picture are also broken down and scored individually, as shown in Table 1. The higher scored factors highlight those risk variables that are worthy of further analysis. The details are not presented here to preserve space, but suffice to say that there are many aspects of corporate strategy that lie beyond the boundary of the firm (Cheah, 2002). Most of these cannot be directly influenced or controlled by the firm, nonetheless they have profound impact on its corporate performance.

Similarly, in Figure 2, the industrial environment of construction may be represented by several key factors. Statistical methods can be used to study some of these key factors. For example, to examine the association between output growth (demand) and profitability, both parametric and nonparametric correlations can be computed using data for 31 geographical regions in China. The results are summarised in Table 2.
Conclusions

Strategic analysis of the Chinese construction industry entails patient analysis and dissection of numerous issues, and requires a blend of strategic management theories and research methodologies. Moreover, the topic is dynamic, as the environment keeps changing. Essentially, this article only presents the tip of an iceberg. In view of an urgent need for Singapore construction players to internationalize, it is pertinent to understand the foreign market environment and problems that even the host country competitors are facing.

References

Applicability of Equivalent Rectangular Stress Block to Nonrectangular Cross-Sections

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In flexural design of reinforced concrete sections, the use of an equivalent rectangular stress block (ERSB) is more convenient than any other representations of the flexural stress distribution of concrete. Hence it is adopted in most design codes. This ERSB is derived from experimental investigations on specimens with rectangular cross-sections and extended for design of members having nonrectangular cross-sections. Figure 1 illustrates the use of an ERSB in ACI code (ACI 318M-02 2002) for different shapes of cross-section. It should be noted that the parameters for ERSB specified in the code have not changed since 1971.

Figure 1. Application of ERSB to rectangular and nonrectangular sections

Previous researchers found that the use of the ERSB is conservative for structural members of normal strength concrete having nonrectangular concrete compression zone. With the use of high strength concrete (HSC), defined as concrete with cylinder compressive strength from 60 MPa to 100 MPa, it is imperative to check the applicability of the ERSB to such a material and nonrectangular cross-sections. While most tests of HSC beams with rectangular cross-sections showed that the use of the ERSB of the ACI code is conservative, results of tests of HSC beams with triangular concrete compression zone by Kahn and Meyer (1995) were lower than predicted by the code. Ibrahim and MacGregor (1996) tested six column specimens with triangular cross-sections under eccentric loading and found that the distances from the resultant force to the neutral axis in all specimens were significantly smaller than those calculated from the code. This means that the predicted moment capacity from the ERSB of the code may overestimate the actual moment capacity of the sections.

An experimental program has been carried out at the School of CEE, NTU on plain and reinforced HSC columns with both square and circular cross-sections to study the flexural behaviour of such members. All the specimens have 800 mm height with either 200 mm width for squared section or 200 mm diameter for circular section. Results from nine circular columns and nineteen square columns tested under eccentric loading were analysed. These specimens were cast from two batches of concrete with target cylinder strengths of 40MPa and 70 MPa, respectively. Test data of each specimen were used to calculate the parameters $\alpha_i$ and $\beta_i$ of the ERSB as shown in Figure 1 in which $\alpha_i$ is the intensity and $\beta_i$ is the depth of the ERSB.

Generally results from specimens in the same batch indicated that the product $\alpha_i\beta_i$ obtained from circular specimens are smaller than that obtained from square ones. The difference is 10% for the normal strength concrete group but can be as much as 23% for HSC specimens. These results also mean that an ERSB calibrated from tests of square specimens would overestimate the load carrying capacity of circular specimens.

Since the ERSB of a stress-strain curve of concrete is dependent on the cross-section shape, a parametric study has been carried out to analytically assess the applicability of the ERSB to nonrectangular cross sections. In this study the stress-strain curve of concrete is assumed to follow the stress-strain model for plain concrete proposed by Collins et al. (1993). Other models capable of predicting the stress-strain behaviour of both normal and high strength concrete should also produce similar results. The ultimate compressive strain is assumed at 0.003 as specified in the ACI code. The ERSB is calibrated from applying the analytical stress-strain curve to a rectangular cross-section.

A numerical integration using 6-point Gauss quadrature was used in the calculation of the internal forces and moments from the analytical stress-strain curves. For the circular section, different ratios between the depth of the neutral axis and the diameter of the section ($c/h$) were considered. The results are depicted in Figure 2 and Figure 3. A value of $r_p$ or $r_M$ is greater than unity means that the force or moment computed from the ERSB is greater than that computed from the analytical stress-strain curve. This also means that the actual capacity of the cross-section is smaller than that computed from the ERSB.

Figure 2 shows that for a triangular concrete section the moment computed from the ERSB is always greater than...
Results from both experimental and analytical assessments show that it may not be compatible to use the ERSB, which is calibrated from tests of specimens having rectangular cross-sections, to predict the capacity of members having nonrectangular cross-sections. It is conservative to use the ERSB for members with triangular or circular cross-sections having concrete strength below 60 MPa but it may be unsafe for higher strength concrete.

References

[1] ACI 318M-02 (2002), Building Code Requirements for Reinforced Concrete and Commentary (ACI 318M-02/ACI 318R-02), ACI Committee 318, American Concrete Institute, Detroit.

Figure 3. Variation of ratios \( r_p \) and \( r_M \) of circular cross-section that computed from the analytical stress-strain curve. The ratio between the former and the latter increases quickly as the concrete strength increases. The value of \( r_p \) for triangular section is not plotted because it is always equal to unity regardless of the shape of the stress-strain curve of concrete. Figure 3 illustrates the variation of ratios \( r_p \) and \( r_M \) with concrete compressive strength in circular sections. Both the load and moment computed from the analytical stress-strain curve are different from those computed from the ERSB. The ratios \( r_p \) and \( r_M \) are higher than unity for HSC and they further increase with the concrete strength. The ratio between the depth of neutral axis and diameter of the section \( (c/h) \) also affects the ratios \( r_p \) and \( r_M \).

**Curvature-Deflection Relationship of Reinforced Concrete Columns Bent in Single Curvature**

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**Introduction**

Deflection of reinforced concrete (RC) columns is important in design practice because it adds an additional moment to the columns due to the \( P \)-delta effect. It is customary to calculate the deflection of reinforced concrete members through the curvature–deflection relationship. This relationship is usually considered to be linear up to the formation of plastic hinges caused by the crushing of concrete in compression or yielding of longitudinal reinforcement in tension. After the formation plastic hinges, the deflection of a column is controlled by the plastic hinge mechanism and is computed based on the equivalent plastic hinge length.

The objective of this research work is to revise the existing formula for the curvature-deflection relationship to take into account the nonlinearity caused by the \( P \)-delta effect of RC columns under eccentric loading. The equivalent plastic hinge length in RC columns under eccentric loading is also proposed.

**Deflection of RC columns before plastic hinging**

For a hinged-hinged column under equal eccentricities at two ends, the moment distribution over the length of the column is not constant due to additional eccentricity from deflection. As a result, the midpoint deflection can be calculated from curvature-area theorems (Kong and Evans, 1987) as follows:

\[
\delta = \frac{1}{8} \varphi_1 l^2 + \frac{1}{K_e} \varphi_2 l^2
\]  

The first part in the right hand side of Equation (1) is the deflection computed from the constantly distributed curvature \( \varphi_1 \) caused by the moments at column ends. The second part is the additional deflection caused by the additional moment due to \( P \)-delta effect, which induces the additional curvature \( \varphi_2 \). \( K_e \) is a parameter dependent on the shape of the curvature distribution. \( \delta \) and \( l \) are the mid-height deflection and total length of the column, respectively.

For instantaneous deflection calculation, it is reasonable to assume that the modulus of elasticity of concrete \( E_c \) is a constant. Hence the ratio between \( \varphi_1 \) and \( \varphi_2 \) can be computed as:

\[
\frac{\varphi_1}{\varphi_2} = \left( \frac{P e}{E_c I_e} \right) = \frac{I_e}{I_e K_e} = K_e e
\]

where \( P \) is the applied load, \( e \) is the initial eccentricity; \( I_e \) and \( l_e \) are the gross moment of inertia and effective moment of
Let \( \varphi = \varphi_1 + \varphi_2 \) be the total curvature at the mid-height section. Expressing \( \varphi_1 \) and \( \varphi_2 \) in terms of \( \varphi \) and rearrange Equation (1) gives:

\[
8K_c\delta^2 + 8\left(eK_rK_e - \varphi^2\right)\delta - eK_rK_e\varphi^2 = 0
\]

(3)

Solving this second-degree equation the solution of \( \delta \) can be found explicitly. Omitting the negative solution, the final expression for the mid-height deflection is:

\[
\delta = \frac{\varphi^2 - eK_rK_e + \sqrt{(eK_rK_e - \varphi^2)^2 + 0.5eK_rK_e\varphi^2}}{2K_c}
\]

(4)

For reinforced concrete columns both the coefficients \( K_r \) and \( K_e \) change during loading due to the reduction of column stiffness in the mid-height region caused by cracking of concrete. For simplification \( K_r \) may be taken equal to 11.0, the value between those calculated from triangle and sine wave curvature distributions, and the effective modulus of inertia \( I_e \) may be taken equal to crack modulus of inertia \( I_{cr} \) to calculate the value \( K_e \) in Equation (2). Test data by Lau and Singh (1994/1995) and Chan and Koh (1995/1996) on slender concrete columns were used to verify the proposed formula. Figure 1 shows that the proposed calculation matched well with the test data.

**Deflection of RC columns after plastic hinging**

It is assumed that after the development of the plastic hinge at the mid-height of the column, the curvature in the length of the plastic hinge increased while the curvature in the other parts remained a constant value evaluated at the onset of plastic hinging. From Figure 2. the mid-height deflection after the formation of plastic hinge can be calculated using curvature-area theorems (Kong and Evans, 1987) as:

\[
\delta = \delta_s + (\varphi - \varphi_s) \int_{l_p} \left( \frac{l - l_p}{2} \right) df
\]

(5)

where \( \varphi \) is the total curvature in the plastic hinge region; \( l_p \) is the equivalent plastic hinge length measured from the section of maximum moment to one side of the column; \( \varphi_s \) and \( \delta_s \) are the curvature and deflection at mid-height section evaluated at the onset of plastic hinging, respectively.

The value of \( l_p \) for RC columns bent in single curvature is evaluated based on test data from the experiment done by the authors. In this experiment, thirteen reinforced concrete specimens under equal end eccentricities were tested. All the specimens were 800mm height with either 200mm square or 200 mm diameter circular cross-section.

A trial-and-error method was used to find the relationship between the equivalent plastic hinge length and the height of the cross-section of each specimen using Equation (5). The results gave the \( l_p \) values in the range of 0.6h to 0.9h with the average value \( l_p = 0.7h \). Figure 3 shows the analytical predicted deflection curves using \( l_p = 0.7h \) plotted along with experimental values for the tested specimens.

**Conclusions**

The deflection-curvature relationship of RC columns has been established. This relationship consists of two parts. The first part is a curve which takes into account the \( P-\delta \) effect. The second part is a straight line starting from the formation of the plastic hinge. The equivalent plastic hinge length was found from tests of the short RC columns under constant end eccentricities to be 0.7 times the section height.

**References**


Moment-Rotation Relationships of Single Fin Plate Connections under Monotonic and Cyclic Loading

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Introduction

The advantage of the single fin plate connection is that they are simple to fabricate and easy to assemble on site. The single fin plate connection produces moment and rotational capacity from the bolt deformation in shear and the fin plate deformation around the bolt holes. This research is to study the behaviour of single fin plate connection with High Strength Friction Grip (HSFG) bolts under monotonic and cyclic loadings. The primary mechanism of HSFG bolts is to prevent the slippage of connected parts while transferring the load.

Modelling of single fin plate connection under monotonic loading

A mechanical model comprising a series of springs is proposed. The column is assumed to be rigid relative to the single fin plate connection and each spring represents the axial response between each HSFG bolt and the connected parts. If the axial force-displacement \( F_s – \Delta_s \) relationship between HSFG bolt and connected parts can be defined, the moment-rotation \( M – \theta \) relationship of the entire connection can be calculated.

From the experimental test, it was observed that the behaviour of single fin plate connection under monotonic loading consists of three typical stages. In the first stage, the \( F_s – \Delta_s \) curve is linear up to the slip resistance \( F_{slip} \). After the slip resistance \( F_{slip} \) is exceeded, the bolt slip occurs without an increase in load capacity. This is defined as stage II. At stage III, the HSFG bolt bears against the connected parts and the relationship at this stage is non-linear and can be given as:

\[
\Delta_s = \frac{F_s}{K_b \cdot \left[ 1 - \left( \frac{F_s}{F_{bearing}} \right)^n \right]}
\]

where \( K_b \) and \( F_{bearing} \) are the initial stiffness and the ultimate bearing capacity of single fin plate connection with bolts in bearing respectively, whereas \( n \) is the shape parameter.

Based on the proposed mechanical model, the moment and rotation of single fin plate connection can be calculated as:

\[
M = \sum_i F_i \cdot y_i
\]

\[
\theta = \frac{\Delta_i}{y_i}
\]

As shown in Figure 1, the predicted results agree well with the test results. For single fin plate connections with different bolt diameter, beam web thickness and plate thickness, the initial bearing stiffness \( K_{bi} \), ultimate bearing capacity \( F_{bearing} \) and shape parameter \( n \) at the third stage can be determined accordingly. Therefore, the proposed model can be extended to simulate the behaviour of single fin plate connection with different configurations.

Modelling of single fin plate connection under cyclic loading

From the test result of specimen under cyclic loading, it was found that the monotonic moment-rotation curve could be used as the envelope curve for the behaviour of single fin plate connection under cyclic loading. The cyclic moment-rotation relationship reflects a combination of cyclic slip and bearing behaviour. Based on the observation of experimental result, the rules of cyclic \( F_s – \Delta_s \) relationship between HSFG bolt and connected parts can be summarized as follows:

- The envelope curve is based on the \( F_s – \Delta_s \) relationship between HSFG bolt and connected parts under monotonic loading;
- Elastic unloading without stiffness change, i.e., the unloading stiffness is the same as \( K_{slip} \);
- Constant elastic range which equals to the twice of the slip resistance \( F_{slip} \).

As shown in Figure 2, when connection is loaded, the initial stiffness \( K_{slip} \) is followed (branch AB). At this stage, the \( F_s – \Delta_s \) curve is linear up to the slip resistance \( F_{slip} \). After the slip resistance \( F_{slip} \) is reached, slip occurs and the displacement increases to \( \Delta_{bearing} \) without any the increase of load capacity (branch BC). When unloading occurs, the unloading stiffness...
is $K_{slip}$ and the slip occurs when the slip resistance $F_{slip}$ is exceeded. The elastic distance is $2F_{slip}$ (branch CD). The load capacity remains constant until $\Delta_{bearing}$ is reached (branch DE). Upon further loading, HSFG bolt bears against the connected parts which results in a non-linear curve, the skeleton curve is followed until unloading occurs at point F (branch EF).

Unloading from F occurs with stiffness $K_{slip}$ and an elastic distance of $2F_{slip}$ (branch FG). The load capacity remains unchanged until the skeleton curve is reached (branch GH). When reloading occurs, the same principle follows (branch HIJK).

Based on the monotonic behaviour of a single fin plate connection and cyclic analytical model, the cyclic moment-rotation relationship of the beam-column connection can be calculated. The predicted results are found to have a good agreement with the test result in Figure 3.

References


**Behaviour of Hybrid Connections under Monotonic Loading**

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**Introduction**

Connection design has always been an important aspect in precast concrete construction. In this study, the ease of steelwork installation is adopted to replace the traditional reinforced concrete connection. This new hybrid connection consists of a single fin plate connection with HSFG bolts plus an in situ conventional reinforced concrete infill. The semi-rigid behaviour of the steel component is complicated further by reinforced concrete infill. A series of experiments were carried out to study the behaviour of hybrid connection under monotonic loading. A component-based mechanical model is proposed to predict the moment-rotation relationship of this hybrid connection under monotonic hogging and sagging moment.

**Experimental programme**

The beam and column sizes adopted in this experimental programme are based on a typical industrial building layout. Three types of specimens were tested to determine the behaviour and contribution of the joint components. The section of one type of test specimen is shown in Figure 1.

A simply supported test arrangement was adopted to determine separately the sagging and hogging moment behaviour of the joint.
for steel reinforcement under monotonic loading. The force-displacement relationship of single fin plate connection with HSFG bolts has also been carried out based on the test results in this study. Taking the rotation of the connection θ as the variable, the analytical model for the moment-rotation characteristics of hybrid connection for both monotonic hogging and sagging moment can be formulated.

The comparison between test results and predictions of the hybrid connections under monotonic hogging and sagging moment are presented in Figures 2 & 3 respectively. Generally, the model predictions agree well with the test results. Using this component-based model, the contribution of each component to the behaviour of the entire hybrid connection can be identified, and this leads to a better insight into the actual behaviour of the hybrid connection.

**Analytical modelling**

A proposed component-based mechanical model is proposed to simulate the moment-rotation relationship of hybrid connection under monotonic hogging as well as sagging moment. Column is assumed to be rigid as represented by a rigid bar. The mechanical model consists of three basic components: single fin plate connection, the longitudinal reinforcements and concrete beam section. The force-displacement (F–δ) relationship of each component under monotonic loading can be defined and represented by an equivalent spring in the model.

Because of the non-uniform deformations along the depth of the concrete beam, the section is subdivided into a number of discrete layers n, each to be represented by a spring. The deformational state of each spring has two components: the axial and rotational. The axial deformation component of a spring element i located at a distance y of the connection can be given as:

\[ \Delta_i = (y_i - \bar{y})\theta \]

where \( \Delta_i \) is the axial deformation component of a spring element i, \( \bar{y} \) is the depth of neutral axis, and \( \theta \) is the rotation of the hybrid connection.

Once the deformation \( \Delta \) is established, the corresponding force \( F_i \) to the connecting element can be derived based on the F–δ relationship of the respective component.

\[ F_i = K_i \Delta_i \]

The total axial force and moment transmitted by the connecting elements can be expressed as:

\[ F = \sum_{i=1}^{n} F_i = 0 \quad \text{and} \quad M = \sum_{i=1}^{n} F_i y_i \]

A neutral axis position has to be determined for each force and deformation state. If the force-displacement relationship of connecting element is inelastic, iterations are needed until the force equilibrium and displacement compatibility conditions are satisfied. The stress-strain relationship proposed by Mander et al. (1988) is adopted in describing the behaviour of concrete. A bilinear model is proposed to present the stress-strain relationship

**References**


Investigating the Load Paths of RC Shear Wall with Openings under Reversed Cyclic Loadings

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Introduction

Currently the application of the walls with irregularly distributed openings is still limited in most countries though these kinds of walls have shown satisfactory performances in the severe seismic regions. Some research [1, 5] also has shown that these kinds of walls could achieve satisfactory strengths and ductility levels. Paulay [3] has proposed a design method using strut-and-tie models for these kinds of walls, which has been used by Yanez [5] to design the wall specimens with irregular openings. However because the presence of openings causes a complex stress state in the wall when it is subjected to the external load, there is still a lack of information to prove that a strut-and-tie model can represent the necessary complex stress state. Therefore, in this study at NTU, an analytical study of these kinds of walls using the non-linear finite element method (FEM) was firstly performed. Then the load paths obtained from the analytical results of stress distributions were discussed and compared with the strut-and-tie models proposed by previous researchers.

Analytical programme

Ten models of structural walls, including six walls (S1~S6) with low aspect ratios identical to those tested by Yanez [5] in the University of Canterbury and the other four slender walls (W-1~W-4) identical to those tested by Ali [1] in the University of Michigan, were analyzed with a non-linear finite element program named WCOM2D. In this program a smeared crack model and a joint element model developed by the University of Tokyo for cracked reinforced concrete were used [2]. Figure 1 shows the meshes of S2, S3, S4 and W-2. The meshes of other specimens were similar to these specimens.

Analytical results of walls

Load-displacement responses

The comparison between the finite element analysis (FEA) result of load-displacement response of each specimen and the experimental one was first carried out. It was found that the two sets of results were very close. That means the program WCOM2D can predict the behaviours of these kinds of walls satisfactorily.

Load paths of the walls with irregular openings (S2~S4)

According to Schlaich [4], the load path method is a fundamental element in developing the strut-and-tie model and the load path can also be achieved in accordance with the mean direction of the principal stresses. Therefore those principle stress flows of specimens S2, S3 and S4, which were designed using strut-and-tie models, were analyzed in this study. Figure 2 shows the principal stress flows of the specimen S2 at the ductility factor (DF) equal to ±1. The blue lines are principal compressive stress while red lines are principal tensile stress. It can be seen that the principal compressive stress concentrated in the diagonal directions of the “panel” or “column” zones of the walls. “Columns” took part in transferring a part of the shear through the diagonal struts in them. Similar principal compressive stress flows were observed in the specimen S3 and S4. Figure 3 shows the load paths of specimen S2 developed according to the principal compressive stress flows. A strut-and-tie model could be developed based on this load path, in which the column’s contribution for resisting shear was considered. The model showed a good correlation with the strut-and-tie model used by Yanez [5] to design and analyze the specimens. Similar strut-and-tie models could be built for S3 and S4.

The load path can also be proved by finding out the proportions of the shear force carried by the column zones. The shear forces in the sections 1-1 and 2-2 (Figure. 3) were analyzed in this study. Figure 4 shows the proportions ($P$) of shear forces transferred by column zones ($V_c$) of specimens S2 ~ S4 to the total forces ($V$) in each cycle. According to the load history, cycle 4 corresponds to the ductility factor (DF) ±1, followed by the 2 cycles for each successive ductility factor. It can be seen that the proportion reached its maximum at cycle 4 or cycle 5 then decreased with the increasing cycles. After cycle 9 (DF=±1), the section 1-1 in the lower column zone of specimen S2 or S3 beside the openings, carried almost no shear force because at this ductility level the concrete was cracked or crushed and a diagonal strut could not form. Conversely the section of specimen S4, due to its larger area, continued sustaining a large proportion of shear force thus enabling the diagonal strut in this zone to be retained. The maximum proportion of shear force carried by the section 1-1 ($P_1$) or section 2-2 ($P_2$) could be approximated by the force equilibrium condition in node A or node B (Figure 3) respectively. It is necessary to satisfy the following equation:

$$P_1 = \frac{V_c}{V} = \frac{\tan \alpha_1}{\tan \alpha}$$  or  $$P_2 = \frac{V_c}{V} = \frac{\tan \theta_2}{\tan \theta}$$
Analysis results of slender shear walls

Similar stress flows were observed in the slender wall. According to the principal stress pattern, the slender walls could also be divided into beam, column and panel zones. Clear diagonal struts were also observed in the panel zones. However, the analysis showed that the column zones beside the openings sustained very little shear force unlike that experienced in the walls with low aspect ratios. The panel zones transferred most of the shear force in the slender walls, whereas the boundary elements sustain only the axial load due to the large compression or tension in them.

Conclusions

The load paths developed based on the principle stress flows show a good relation with the strut-and-tie models proposed by previous researchers. According to the load path, a lateral force was transferred to the foundation by diagonal struts in the column and panel zones. A strut-and-tie model considering the column contribution to resist shear could estimate the maximum shear strength of the walls with irregular openings, while a model neglecting the contribution of column zones could be applied to evaluate the shear capacity of the wall at high ductility levels. In the slender walls with staggered openings, the zones in the web transferred most of the shear force to the foundation.

References

A Theoretical Approach to Determine Strut Angles of a Variable Angle Truss Model for Reinforced Concrete Beams

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Introduction

Most of current shear design procedures for reinforced concrete beams are largely based on a truss model analogy. At the failure stage, cracks form in the reinforced concrete beams. The inclined concrete struts represent the concrete blocks between adjacent cracks and transfer external loads to ties connected to them, which are the shear reinforcement in the transverse direction and are represented as lumped areas of reinforcement crossed by the struts. The ties act as the component for shear resistance under this truss model analogy. Top and bottom chords consist of concrete compressive stress blocks and longitudinal reinforcement respectively.

Strut angle, which represents the angle of diagonal compression, is a key parameter in this shear design philosophy. For simplicity in dealing with strut angles, 45° was first assumed corresponding to the first shear cracking angle. However, this assumption may lead to a low contribution from the shear reinforcement especially for the shear strength of lightly reinforced concrete beams. Other researchers then have suggested taking 30° as the struts inclination. The observation found in these proposals was that one constant strut angle was assumed over the entire shear span of beams. This may be erroneous as past experiments always showed cracks orientated towards different directions at different regions along the shear span of a beam, indicating the varying directions for diagonal compression. On this basis, variable angle truss models have been conceptually developed in some research. However, the exact variable strut angles have not been given in these conceptual models in mathematical solution or mechanical approach.

This research herein tries to develop a more general approach to the evaluation of strut angles of a variable angle truss model for reinforced concrete beams subjected to shear.

Evaluation of strut angles of a variable angle truss model

Shear transfer mechanism for a typical region along a beam member can be reasonably represented by a truss model analogy as shown in Figure 1. Under this analogical assumption, a few truss units can be formed along the shear span of a cracked “long” beam member. In each truss unit, the inclined diagonal strut transfers the shear force to the vertical tie, and the tie resists it. At the same time, the top and bottom chords are responsible for flexural resistance. By deconstructing the beam shear span in this manner, it is found that the rigidity and stiffness are more easily accessed for each truss units. The rigidity and stiffness of the truss unit are the summations of all the members in the unit. Then the deformation of each truss unit can be computed. Furthermore, external work done to each truss unit can be determined. This allows the strut angle to be examined by minimizing the external work done. The Principle of Virtual Work is employed as the basic analytical tool in this part of the paper.

When applying the Principle of Virtual Work on each truss unit, the axial rigidity of each member forming the truss unit is the most important and must be studied with care. The typical truss unit subjected to a shear force $V_s$ as shown in Figure 1 is considered again. Firstly, it is assumed that the transverse shear reinforcement is uniformly distributed over the length of the member. Under this smeared shear reinforcement assumption, the axial rigidity of the tie is:

\[ (EA)_t = \cot \theta \rho_s n E_s A_s \]

For the inclined strut, the area is determined from geometrical consideration. Conventionally, it is taken as:

\[ (EA)_s = b_s jd \cos \theta \]

Then the axial rigidity of the strut is:

\[ (EA)_s = b_s jd \cos \theta \cos \theta E_s = \cos \theta E_s A_s \]

For the flexural members, it is assumed that the bottom tensile member is placed at the centroid of the bottom longitudinal reinforcement, while the top compressive member is at the centroid of the concrete compressive stress block. Hence, the height of the truss is the internal level arm $jd$. A distinction in axial rigidity should also be made between
the bottom tensile member and top compressive member. For the tensile member, the concrete around its position is normally cracked and does not contribute significantly to the axial rigidity of those members compared with the reinforcement there. On the other hand, in addition to the concrete compressive stress block, there is a rigidity contribution from the top longitudinal reinforcement for the top compressive member. Generally, the centroid of the top longitudinal reinforcement differs from that of the concrete compressive stress block. In this paper, the top longitudinal reinforcement is simplified to be at the same position of the centroid of the concrete compressive stress block. This simplification might cause a slightly greater deformation of the top longitudinal reinforcement than the actual condition if the centroid of the concrete compressive stress block is deeper than that of the position of the longitudinal reinforcement. Thus, the axial rigidity of the bottom tensile member is:

\[ (EA)_t = E_A A_t = \rho_n E_A A_t \]

For the top compressive member, the axial rigidity should be:

\[ (EA)_c = \left( \frac{c}{d} + \rho_s(n-1) \right) E_A A_t \]

These two equations describe the dimensioning of the top and bottom chord members.

Member forces of the truss unit can easily be found by static equilibrium. Then the deformation of the truss unit due to applied shear force can be calculated using the Principle of Virtual Work. A summary of the analysis is presented in Table 1. The deformation of the truss unit is the sum of the member deformations, thus,

\[ \Delta = \sum_{i} F_{i} \frac{\theta_{i}}{EA} = \frac{1 + \rho_n n \cos \theta E_A A_t}{\rho_n n \cos \theta E_A A_t} j d V_s + \left( \frac{l}{jd} - \cot \theta \right)^2 \left( \frac{c}{d} + \rho_s(n-1) \right) E_A A_t \]

\[ + \left( \frac{l}{jd} \right)^2 \cot \theta - j d V_s \]

The deformation of the truss unit is formatted in such a way that the first term in Equation (6) is the deformation contributed from shear members (strut and tie) and the second term is from the flexural members. The drift angle is determined by dividing the deformation by the length of the truss unit, thus,

\[ \alpha = \frac{\Delta}{jd \cot \theta} = \frac{1 + \rho_n n \cos \theta E_A A_t}{\rho_n n \cos \theta E_A A_t} V_s + \left( \frac{l}{jd} - \cot \theta \right)^2 V_s + \left( \frac{l}{jd} \right)^2 \frac{c}{d} \frac{\rho_s(n-1)}{E_A A_t} V_s \]

\[ + \left( \frac{l}{jd} \right)^2 \frac{\rho_n E_A A_t}{jd} \]

It is assumed that the diagonal compression will develop in the orientation that requires a minimum amount of external energy. Hence, the angle \( \theta \) that can minimize Equation (9) is the strut angle. Thus by differentiating the equation with respect to \( \theta \) and minimizing the external work done, the strut angle causing the minimum energy can be derived as:

\[ \left( \frac{l}{jd} - \cot \theta \right)^2 \cot \theta \left( \frac{c}{d} + \rho_s(n-1) \right) E_A A_t + \left( \frac{l}{jd} \right)^2 \cot \theta \frac{\rho_n E_A A_t}{jd} \]

\[ \frac{dEWD}{d\theta} = 0 \]

Carrying out the differentiation of Equation (10) leads to the following solution for the crack angle \( \theta \):
This equation is a four degree-one variable equation about \( \theta \). An analytical solution can be found for this equation; however, a trial and error procedure to get a solution of the strut angle \( \theta \) is good enough.

The solution of the strut angle varies along the shear span of the beam as the variable \( l \), which represents the available span length, is different for each truss unit. For a particular shear level, the solution procedure starts from the loading point and moves towards the support in a shear span. According to Equation (11), strut angle \( \theta \) for the first truss unit can be found by substituting the total shear span length \( a \) to the variable \( l \). With this \( \theta \) value, a check of \( jd \cot \theta \) which represents the length of this unit truss can be done. If the result shows that \( jd \cot \theta \) is smaller than \( a \), the solution procedure should continue for next truss unit by updating the variable \( l \) with a new value \((a – jd \cot \theta)\). Then strut angle \( \theta \) for the next truss unit can be obtained with Equation (11) again. The process stops when the check shows that variable \( l \) used to calculate strut angle \( \theta \) for a new truss unit is smaller than the length \( jd \cot \theta \) of this newly formed truss unit (available span length is not enough for a new truss unit). So the solutions of the strut angles for truss units along the shear span of the beam differ in a decreasing manner when moving towards the support as the variable \( l \) gets smaller. Moreover, when the shear increases, a few variables such as \( c \) in Equation (11) are also affected, and hence the solutions of the strut angles are different. Thus, a continuous profile of struts orientation development can be found in this analysis. Figure 2 shows the results of this analysis at the ultimate stage of a reinforced concrete beam. The beam and the crack pattern were extracted from Bresler & Scordelis.

**Conclusion**

In summary, this theoretical method has two distinct characteristics for the evaluation of the strut angle \( \theta \). Firstly, the strut angles calculated from this method are different along the shear span from the load point to the support (different strut angle for different truss unit). This effectively develops a variable angle truss model for reinforced concrete beam. Secondly, the angles can vary with the increase of the shear force level. Thus the change of direction in the development of the diagonal compression can be seen. It matches with the crack patterns observed in most of the tested reinforced concrete beams. This method developed here is the foundation for further analysis of reinforced concrete beams subjected to shear with a variable angle truss model.

**References**

Electromagnetic Properties of Cement Paste

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Introduction

Many developed countries currently dedicate a considerable portion of the construction budget for restoration, repair, and maintenance of old structures as opposed to new construction. The anticipated economic impact of an extensive infrastructure repair scheme has produced a renewed interest in improving non-destructive testing methods for assessing concrete structures. Among the few available methods, electromagnetic techniques have shown increasing potential for the rapid evaluation of concrete condition in situ. However, interpretation of data produced by these techniques requires a clear understanding of the relationship of the electromagnetic properties of concrete to the physical properties of interest (e.g., porosity, moisture content, chloride content, etc.). The objective of the research is to establish such relationships. A mathematical model has been proposed to establish those relationships. However, in this article, only the experimental results are presented and discussed.

Experimental work

The cement paste cubes made of various water/cement ratios and air-entrained-agent contents were cured in water for 120 days, at room temperature 20 ± 3°C to ensure full hydration. Then, the specimens were stored in four different conditions to obtain four different moisture states as shown in Table 1.

<table>
<thead>
<tr>
<th>Moisture State</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet</td>
<td>Submerged in water, at room temperature</td>
</tr>
<tr>
<td>Saturated - surface dry (SSD)</td>
<td>Stored in curing room at room temperature for 28 days, with relative humidity higher than 90%</td>
</tr>
<tr>
<td>Air-dry (AD)</td>
<td>Exposed to ambient room temperature and relative humidity for 28 days</td>
</tr>
<tr>
<td>Oven-dry (OD)</td>
<td>Stored in the oven, at temperature 105°C, for 48 hours (until constant weight).</td>
</tr>
</tbody>
</table>

The oven-dried samples were immersed in solutions with four different chloride concentrations (0.5%, 1%, 2% and 4%Cl⁻) for 3 months. The porosity of cement pastes were measured by Mercury Intrusion Porosimetry and the electromagnetic properties were measured by Open-Ended Coaxial Probe method at the frequency range from 0.2 to 6.0 GHz. The set up of the equipment and the arrangement of the specimen are shown in Figures 1 and 2.

Results and discussions

Experimental results to describe the relationship between the electromagnetic properties of cement paste and the physical properties of interest (e.g., porosity, moisture content, chloride content, etc.) are presented in this section.

Effect of void content on dielectric constant of cement paste

For oven-dried cement paste, the higher the void content (capillary porosity and air content) is, the lower the dielectric constant. It is due to the lower dielectric constant of air compared to solid materials. Figure 3 shows the relationship between water/cement ratio and dielectric constant of oven-dried cement paste. The higher water/cement ratio, which forms more porous cement paste (as shown in Table 2), will lead to the lower dielectric constant. Therefore, dielectric
constant (electromagnetic property) of oven-dried cement paste can be used to indicate its quality.

Effect of moisture content on dielectric constant of cement paste
Increase of moisture content in cement paste will increase its dielectric constant (as shown in Figure 4). It is due to the fact that the dielectric constant of water is much higher compared to solid materials and air. Therefore, the dielectric constant of cement paste may indicate its moisture state. Since, the moisture state of cement paste is strongly related to its deterioration, the dielectric constant may indicate the durability of cement paste.

Effect of chloride concentration on dielectric constant of cement paste
The presence of high concentration chloride ion in concrete surrounding reinforcing bars will initiate the corrosion. Therefore, it is very important to limit chloride concentration in concrete. Figure 5 shows that the dielectric constant of cement paste on SSD condition is affected by chloride concentration. The effect of chloride concentration on the dielectric constant of cement paste is more obvious at low frequency. Therefore, dielectric constant can be used to predict the chloride concentration in existing concrete structure.

### Table 2. Capillary porosity and air content of cement pastes with various water/cement ratios

<table>
<thead>
<tr>
<th>Water/Cement Ratio</th>
<th>Capillary Porosity (%)</th>
<th>Air Content (%)</th>
<th>Capillary + Air Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>10.62</td>
<td>1.8</td>
<td>12.4</td>
</tr>
<tr>
<td>0.4</td>
<td>14.79</td>
<td>1</td>
<td>15.8</td>
</tr>
<tr>
<td>0.5</td>
<td>17.87</td>
<td>0.8</td>
<td>18.7</td>
</tr>
<tr>
<td>0.6</td>
<td>26.18</td>
<td>0.5</td>
<td>26.7</td>
</tr>
</tbody>
</table>

Conclusions
The relationship of the electromagnetic properties of cement paste to the physical properties of interest (e.g., porosity, moisture content, chloride content, etc.) has been shown by experimental results. Such relationships are to be established quantitatively so that the electromagnetic techniques can be adopted in Non-Destructive Test methods to predict the quality of existing concrete structures.

References
**BLIS-XML Based Information Server for Collaborative Building Design**

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**Introduction**

A BLIS-XML based information server has been developed in this research for web-enabled collaborative building design. J2EE technology has been chosen for the implementation of the server, the web and XML technologies are used for the collaboration and information sharing between different disciplines in the AEC/FM (Architecture, Engineering, Construction/Facility Management) industry. The IFC (Industry Foundation Classes) has been adopted as the information model of the server to facilitate the interoperability. A native XML DBMS (Database Management System) has been chosen to store the BLIS-XML data in the server. The current implementation of the server system mainly supports the collaboration between architectural design and structural design, such as the transformation of design model contents and representations from the architectural domain to the structural domain, and remote visualization and interaction by the Java3D technology. A case study is presented to illustrate the use of the information server for architectural and structural design collaboration.

**Software architecture**

Figure 1 illustrates the architecture of the software system. It follows a three-tier model: client, server and data storage. Currently, the client tier only supports Java applet enabled web browsers.

Inside the server, there are three layers: the data access layer, business logic layer and web component layer. The data access interface is for reading the data in the data storage.

Business logic beans contain all the algorithms for processing geometric information. The web component contains JSP (JavaServer Pages), which dynamically generate web pages. This part of the design follows the popular Model-View-Controller (MVC) architecture, which is illustrated in more detail in the class diagram (Figure 2).

The Tamino XML database was chosen for data storage instead of normal RDBMS (relational database management systems) because of Tamino’s ability in storing XML data natively. It has an enormous advantage over RDBMS. The table-based data model of the RDBMS does not suit the hierarchical and interconnected nature of XML objects. An RDBMS would need to break an XML document into a multitude of interrelated tables. A query against this database would result in much relational retrieval and many joint operations, requiring a high processing power to overcome a considerable degradation of performance. A native XML storage is the essential method to avoid performance limitations that are crucial in our model server.
Case illustration

The architect first logs in the web site using Internet Explorer 6.0 in a remote computer (Figure 3). The server checks the identification and authorization of the architect.

![Figure 3. Screenshot of architect logs in JSP page using Internet Explorer on a client PC](image)

Figure 3. Screenshot of architect logs in JSP page using Internet Explorer on a client PC

If the engineer agrees to the structural model, the server will generate a web page to close the information feedback process and allow the architect and structure engineer to download the architectural model as an IFC data file and the structural model as an XML format file. The output structural analysis model file is in XML format.

As the output is in XML format, it can be easily transferred to other data formats required by the structural analysis software. The current IFC2.x Release does not include the structural analysis domain. Thus, the calculated structural model cannot merge with the IFC model. This situation could change in the near future with the unifying work being done by the Structural Domain Group of the German IAI Chapter. [International Alliance for Interoperability (IAI) is the organization for IFC development.]

Conclusions

With the development of Industry Foundation Classes and network technologies, it has become possible for closer design collaboration between architects and structural engineers. The implementation of the IFC-based server proves that the structural model can be derived programmatically from an architectural model. The efficiency of design process can be improved by using the network technology. Also, the Java technology has been proven to be suitable for offering the desirable functionalities required by the collaborative design process.

The structural engineer in any location can log in (by using Netscape Navigator in this case) and view the structural model in his browser. Though he cannot directly change the structural model, he can pick any element and add new properties or comments (Figure 5).

![Figure 4. The architecture of a 7-storey building as the input data](image)

Figure 4. The architecture of a 7-storey building as the input data

The structural model in Java applet allows the client engineer to zoom in, zoom out and walk through the structural model to have a close-up view. The pop-up window has the information about the dimension of the picked element and the location of the idealized structural element and deduced joints. It also allows the structural engineer to add some new properties to the picked element as name-value pairs and send back the information. If the engineer agrees to the structural model, the server will generate a web page to close the information feedback process and allow the architect and structure engineer to download the architectural model as an IFC data file and the structural model as an XML format file. The output structural analysis model file is in XML format.

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Assessment of IFCs (Industry Foundation Classes) for Structural Analysis Domain

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Introduction

As the most widely accepted and supported standard data model by the AEC/FM industry, Industry Foundation Classes (IFC) have made substantial progress in recent years. The aim of this collaborative research project was to verify and implement the improved IFC models by developing a model server which carries out the information exchange between the architectural design and structural design applications. During the course of the project, two practical structural analyses and architectural design software, SAP2000 and ArchiCAD were selected as the examples to be integrated. SAP2000 can not support the IFC file format, so translation between the different formats is needed. During the translation, it was found that some information was defined in different ways which may be inferred from the data in IFC models while some information did not have corresponding definitions in the current IFC models. This is probably because IFC models are more developed from an expert’s perspective rather than from a software application user’s perspective. In order to implement the integration, it is necessary to assess the usefulness of current IFC models to meet the practical needs of structural analysis applications.

Scope of project

Data exchange between architectural design and structural design is a broad and complex process. Since the architecture information has been maturely defined in IFC and some architectural design software packages are able to generate IFC files, the focus of the project was on the structural design process. However, the initial planning, which included a rough specification of design requirements, was assumed to occur prior to conceptual design and was outside the scope of the study. Structural analysis was limited to reinforced concrete design using SAP2000. Considering the complexity of information needed, dynamic analysis was not within the scope. For architectural design, the software used was ArchiCAD 7.0.

Identification of SAP2000’s information requirements

Based on the process models, all the information needed by SAP2000 to perform structural analysis can be classified into 5 different categories by their functions: (1) geometry information (2) material information (3) load information (4) member section information, and (5) other information. The advantage of these classifications includes making information clearer because similar functions should provide common characteristics. The overview of the five different categories and their corresponding information or properties is shown in Figure 1.

Assessment of IFCs product model for structural analysis domain

In order to review the capabilities of IFC product models to support structural analysis at multiple levels of detail, a series of comparison between the information requirements and the current IFC models was done. The findings with respect to the suitability of the IFC Release 2.X Edition 2 to support structural analysis processes are as follows:

- For a simple frame structure, most of the mechanical features and properties necessary for static structural analysis can be found in the current IFC release.
- Some information needs to be inferred from the data in IFC.
- However, the only items IFC does not support include prestress load, description for stresses and types of load combinations.

Principle of IFC extensions development and generality studies for information gaps

In some scenarios, more than one kind of approach can be adopted for IFC extension model development. Adding new classes or additional attributes to the existing classes will impact the exchange files previously generated in current IFC implementation activities. So the principle to IFC extension model development is the attempt to use the same attribute names and definitions already existing within the IFC Model to express similar ideas. This minimizes conflict and confusion for organizations that will implement the extension model. In order to confirm whether it is necessary to make any changes to the current schema, Generality Studies are implemented through investigating six other structural analysis products. Investigation can determine if SAP2000 defines information in a common way. We can then decide the final solution for these gaps and develop the corresponding
extensions. The detailed table of investigation is not shown here but the primary results of Generality Studies are as follows:

(1) Geometry
- The ways to define the “Restraints” of Joint or “Release” of frame elements in all software are the same. They can be inferred from IfcBoundaryNodeCondition for IfcStructuralConnection or IfcRelConnectsStructuralMember. So this information can be considered to be added into the current IFC by a Property Set.
- The concept of Joint Pattern is specific to SAP2000. It is not a general concept among others. Therefore it is not necessary to provide a new class for this concept. We can handle this during programming.

(2) Load
- “Prestress Load” is not within the scope of project ST-4, an IFC project on steel design standards. It is obvious that new class is needed as a subtype of entity IfcStructuralLoadStatic.
- “Combo type”: We can add this new attribute to Entity IfcStructuralLoadGroup in future uses of the IFC model.

(3) Material
- In the current IFC Release only isotropic materials are allowed. In future releases, anisotropic materials and their usage may be considered. It means that new attributes or new classes that are subtypes of the existing classes IfcMaterial are needed.

Recommendations for IFC extension model development

The above section’s discussion for information is all from software viewpoint. From an information technology viewpoint, the extension developments for the above primary information gaps are as follows:

(1) Expression of restraints of joint or release of member through use of property sets
For concepts existing in the IFC model, property sets are used. This approach has the advantage of coexistence and cooperation with other domain’s schemas. Figure 2 shows the expression of structural items in IFC and the relationship with the extension.

(2) New class for prestress load
Prestress load belongs to new concepts. A new class is required to extend the IFC load resource. The definition suggested for prestress load is shown in Figure 3.

(3) Combo types & types of material
In current IFC models, the type of load combination is always set as additive (where the value is “ADDY” in SAP2000) by default. The other three types are ENVE, ABS and SRSS, ENVE is used for moving loads. ABS and SRSS are used for lateral loads. Both the combo types and material types are concepts which extend the IFC model. That is, they need extension to fully capture additional information requirements. Figure 4 illustrates the new attribute added to class IfcMaterial for describing the material types.

Conclusions

In this paper, the information requirements of SAP2000 are analyzed from the software point of view. After comparing the information requirements with the current IFC models, it can be found that most data needed by SAP2000 for structural analysis are explicitly supported by current IFCs. However, IFCs still do not provide representation for some information, such as prestress load and types of load combination etc., which means that some improvements to current IFCs may be necessary. Thus, some suggestions on IFC extensions model development for these information gaps are proposed. Of course, they may just represent some problems which exist between definitions of software and IFC models during implementation. These requests will have some influence on current release of IFC and will need to be further reviewed by the International Alliance for Interoperability (IAI), the organization for IFC development.
Differences in Smoke Spread in Staircase and in Stairwell

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Introduction

In this study, a building is assumed to be attacked by a vehicle bomb at ground level. A car containing 1 ton of TNT is detonated at certain distance away from the building facade. The lower-storey car park was within the diameter of the fire ball generated due to the explosion of 1 ton equivalent TNT. Cars parked in different storeys are assumed to be ignited by explosion-induced fireballs. Currently, modelling of smoke spread throughout the entire building is not feasible since the scale and geometry are very large and complicated. This study only focuses on smoke spread within an exit staircase using software FLUENT. As shown in Figure 1, 15 storeys are modelled and in the staircase, the floor-to-floor height is 4.2 m (Pan, 2003).

Numerical simulation

The burning cars in the car park are converted into wood-equivalent fuel to simplify the problem. In this case study, the heat release rate (HRR) of a typical car fire is assumed to follow formula (1) (Persson, 2002).

\[
Q(t) = \begin{cases} 
αt^2 & (t \leq 20 \text{ minutes}) \\
8.0 & (20 < t \leq 40 \text{ minutes}) \\
Q_{\text{max}}e^{-\beta\left(t - \frac{Q_{\text{max}}}{\alpha}\right)} & (t > 40 \text{ minutes})
\end{cases}
\]

where \(α = 0.0056\text{kW/s}^2\) and \(β = 0.0007\text{s}^{-1}\).

From Figure 2, it can be noticed that \(Q(t)\) follows the \(t^2\) formula, then remains constant at 8 MW for 20 < t < 40 minutes. Thereafter, the heat release rate decreases to zero exponentially.

Criss-crossing stairs in the staircase make CFD modelling very tedious. So in this study, a stairwell without stairs was initially analyzed to study the effect of openings on smoke flow between stairwell (without stairs modelled) and staircase. If the correlation is positive, the results of stairwell can be used to demonstrate the principle of smoke flow in staircase, and subsequent parametric studies can be greatly simplified. This was found not to be the case.

During the computation, it is found that the vertical opening area of exit door has a considerable effect on the air flow inside both staircase and stairwell. To investigate the opening effect on toxicity level, 4 cases of staircase and 3 cases of stairwell were modelled according to a specified percentage of floor plan as vertical openings. Four cases of staircase are: 1) Exit door in each storey is totally closed except at ground level. 2) Exit door is 1/3 open (4% of floor plan of exit staircase as vertical opening). 3) Exit door is 2/3 open (8% of vertical opening). 4) Exit door in each floor is totally open (15% of vertical opening). Similarly, three cases for stairwell are classified according to totally open, 1/3 open and totally close, respectively.

Some interesting phenomena are observed in the numerical simulation of smoke movement. In the study about the effect of opening on smoke flow in staircase, it is found that a larger opening leads to a longer time in the rise of smoke to the topmost floor. As shown in Figure 3, if all exit doors are closed, 11 storeys will be filled up by wood volatiles after 2500 seconds. However, if all exit doors are completely open, wood volatiles can only reach the 5th storey after 2500 seconds. On the other hand, the behaviour of smoke flow in stairwell without stairs is very different. A larger opening causes more rapid smoke spread in the vertical direction. In Figure 4, if all exit doors are closed,
smoke can spread up to 11th storey after 2500 seconds. For the same period, if all doors are open, smoke can fill up to 14 storeys, but the density of smoke is lower than the other two cases.

Figure 5 and Figure 6 are used to explain the two apparently contradicting phenomena. In the lower part of the stairwell (Figure 5), the upward smoke flow is smoother and relatively faster. This means that the internal pressure is lower as compared with the external pressure from classical Bernoulli equation. Thus, in the lower zone of the stairwell, external air is drawn through openings which helps to further accelerate the upward smoke movement. At the top of the stairwell, since the smoke flow is obstructed, upward velocity is close to zero and there is a corresponding built-up of internal pressure. This in turn drives the smoke out of the upper part of the stairwell. Overall, the smoke flow is fairly streamlined on the left side of stairwell without openings.

The above explanation can be verified by numerical results from FLUENT on both stairwell and staircase models. In Figure 7 and Figure 8, both the upward velocity and the static pressure of smoke flow versus elevation are respectively plotted at $t=250s$.

When stairs are modelled as in Figure 6, smoke flow is heavily obstructed by the criss-crossing stairs and consequently, the upward velocity of smoke is slow. Firstly, the smoke fills up the volume of ground storey, then spreads upwards to the second storey. In the second storey, due to obstruction, smoke cannot easily flow up to the third storey, leading to a built-up of internal pressure. Once the internal pressure is greater than the external pressure, the opening at that floor level will allow the built-up smoke to escape. At the same time, some smoke will spread to upper storeys and then fill up the volume of the third storey. Thus, the same in-filling of smoke is repeated from floor to floor. During this procedure, as shown in Figure 9, the smoke flow is no longer smooth and streamlined as in the stairwell. The velocity of smoke varies from positive to negative sign within the staircase indicating some smoke particles actually descend. That means the smoke flow has created a lot of vortices in the flow field. Figure 10 shows that the internal static pressure is positive at most CFD nodes, confirming that the internal pressure is greater than the external one. Hence, a larger opening leads to a greater smoke discharge at an individual storey. Relative to the stairwell case, this leads to a much slower smoke rise to the top storeys.

From the comparison between the smoke flow in staircase and that in stairwell, one can conclude that the two kinds of smoke flow are entirely different. Thus, it is not appropriate to simplify the smoke flow problem in staircase to that of stairwell. Instead, the CFD model should include criss-crossing stairs for more realistic simulation of smoke rise.

Conclusions

In this paper, smoke spread in staircase and stairwell are modelled by FLUENT. Numerical results indicate that the two kinds of smoke flow are entirely different. In staircase, a large opening is helpful to slow down the upward smoke spread. However, in stairwell where there is no staircase, when the wall opening becomes larger, the upward velocity of smoke actually becomes higher. Another observation is that there is an inverse relationship between the smoke density and the opening size for smoke flow in staircase, that is, the greater the opening size, the smaller is the smoke density in the staircase.

Acknowledgments

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References


Tenability Analysis in Dynamic System

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Introduction

It has been reported that most fire victims die from smoke inhalation rather than from burns. However, there have also been some controversies as to how to characterize the potential harm arising from inhaling toxic smoke. Fortunately, it is now possible to perform computations of fire hazards using the term LC50, by which to describe the toxic potency of fire smoke (Richard G. Gann, et. al., 1994). In the calculation, the measured quantity LC50 is the constant concentration (g/m³) of smoke which is lethal to 50% of the exposed test animals in a specified duration. The value of LC50 is obtained using the animal tests performed on bench-scale apparatus with a fixed concentration of smoke. Once LC50 is obtained, it can be used in the tenability analysis for human beings. However, some manipulations of the equations are required, as smoke concentration in real fires varies gradually with time.

Formulation

Not only the concentration index, the LC50 must also include the exposure period. Since people are likely to inhale smoke from real fires for different periods of time, it is necessary to obtain appropriate LC50 values for different exposure times. Experimental data from 5 to 60 minutes exposure to CO/CO2 concentration can be summarized from the following (Richard G. Gann, et. al., 1994):

\[ LC_{50} = M_0 \cdot \frac{1}{F} \]  

where \( M_0 = 45.0 \text{ g/m}^3 \), \( LC_{50} \) is the potency of smoke in g/m³, and \( t \) is the tenability duration in minute.

Hence, with regard to a specific value of \( LC_{50} \), the tenability duration \( \hat{t} \) can be derived as:

\[ \hat{t} = \left( \frac{M_0}{LC_{50}} \right)^2 \]  

Denoting the equivalent mole weight of smoke by \( W_{\text{smo}} \), equation (2) can be expressed in terms of mole fraction of wood-volatile \( F_w \) since \( F_w = 0.0224\cdot\frac{L_{50}}{W_{\text{smo}}} \):

\[ \hat{t} = k^2 \cdot \frac{1}{F_w^2} \]  

where \( k = 0.0224\cdot\frac{M_0}{W_{\text{smo}}} \).

It must be noted that Eq. (3) is only valid for a static system with a constant \( F_w \) and it fails in a dynamic system where \( F_w \) varies with time \( t \). In this article, a simple approach is proposed to calculate the tenability duration from \( t=0 \), as demonstrated in Figure 1.

First of all, at time \( t_m \) when mole fraction is measured, the corresponding tenability duration \( \hat{t}(t_m) \) is calculated according to Eq. (3):

\[ \hat{t}(t_m) = \frac{k^2}{F_w^2(t_m)} \]  

Clearly, Eq. (4) is only valid for the specific case when people enter the staircase at time \( t_m \), and mole fraction of smoke is kept constant at the level of \( F_w(t_m) \). However, for people who are already exposed to toxic smoke from \( t=0 \) as in the dynamic system (Figure 1) over a period of time, \( t_m \) has already lapsed before \( \hat{t}(t_m) \) is calculated. Thus, the real tenability time must be shorter than \( \hat{t}(t_m) \), since a period of time (denoted by \( \Delta t \)) must be subtracted from \( \hat{t}(t_m) \). Obviously, \( \Delta t \) since \( F_w \) varies with time. For instance, if mole fraction of smoke is always smaller than \( F_w(t_m) \) within \((0, t_m)\) and \( \Delta t > t_m \) on the other hand. However, \( t_m \) can be converted into an equivalent \( \hat{t}(t_m) \).

In Figure 1, a very small time increment \( \Delta t \) which is located at the time axis can be considered. Since \( F_w(t) \neq F_w(t_m) \), \( \Delta t \) cannot be directly subtracted from \( \hat{t}(t_m) \), it must be converted into an equivalent time. Denoting the equivalent time by \( \Delta \hat{t} \), one can derive Eq. (5) based on the relationship in Eq. (3):

\[ \Delta \hat{t} = \frac{F_w^2(t_m)}{F_w^2(t_m)} \cdot \Delta t \]  

Hence, \( t_m \) can be further calculated:

\[ t_m = \int_{0}^{\hat{t}(t_m)} \frac{F_w^2}{F_w^2(t_m)} d\tau \]  

Therefore, denoting by \( t_m \), the real tenability duration should satisfy the following equation:

\[ \hat{t}(t_m) - t_m = 0 \]  

Substituting (4) into (7), one obtains:

\[ \int_{0}^{\hat{t}(t_m)} F_w^2 d\tau = k^2 \]
$t_{ten}$ can then be evaluated once $F_w(\tau)$ is obtained.

**Numerical example**

Using FLUENT, transient analyses of species transfer have been conducted for a staircase subjected to a car park fire. As shown in Figure 2, this staircase is 15 storeys high where the floor-to-floor height is 4.2m.

![Figure 2. CFD model of staircase](image)

![Figure 3. Mole fraction of wood-volatile in staircase](image)

Mole fractions of wood-volatile of all respective CFD nodes at 5, 10, 15, 20,..., 50 and 55m elevation, are averaged and plotted in Figure 3. It can be seen that the mole fractions of wood-volatile depend on both the elevation and the time. In this example, for elevation $h$ within the range from 5m to 35m (away from the boundary effects at the top and bottom part of the staircase), mole fractions of wood-volatile can be represented by a piecewise-defined function as follows (Tan et al., 2003):

$$F_w = \begin{cases} 
0 & t < t_1 \\
(t-t_1)/700 & t_1 \leq t < t_2 \\
1.0 & t_2 \leq t 
\end{cases}$$

(9)

where $t_1$ and $t_2$ are functions of elevation:

$$t_1 = 35h + 375, \quad t_2 = t_1 + 700, \quad 5m \leq h \leq 35m$$

(10)

In this example, it is assumed that $W_{nox}=31.4g/m_3$ and $k=0.25$ if tenability duration is measured in second. Applying (9) for $F_w$, the real tenability duration in the staircase can be evaluated by (8). Clearly, $t_{ten}$ depends only on the elevation height. In Table 1, the tenability periods $t_{ten}$ under the most severe situation (when all exit doors are close) are listed (Tan et al., 2003).

As an example, consider $h=25m$, $t_1=1250s$ and $t_2=1950s$ are then obtained according to (10). Substituting (9) into (8), one gets $(t_{ten}–1250)^3=91875$. Therefore, $t_{ten}=1295s$.

**Table 1, Tenability time at different elevation**

<table>
<thead>
<tr>
<th>$h(m)$</th>
<th>$t_1(s)$</th>
<th>$t_2(s)$</th>
<th>$t_{ten}(s / min)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>550</td>
<td>1250</td>
<td>595 / 10.0</td>
</tr>
<tr>
<td>10</td>
<td>725</td>
<td>1425</td>
<td>770 / 12.8</td>
</tr>
<tr>
<td>15</td>
<td>900</td>
<td>1600</td>
<td>945 / 15.7</td>
</tr>
<tr>
<td>20</td>
<td>1075</td>
<td>1775</td>
<td>1120 / 18.7</td>
</tr>
<tr>
<td>25</td>
<td>1250</td>
<td>1950</td>
<td>1295 / 21.6</td>
</tr>
<tr>
<td>30</td>
<td>1425</td>
<td>2125</td>
<td>1470 / 24.5</td>
</tr>
<tr>
<td>35</td>
<td>1600</td>
<td>2300</td>
<td>1645 / 27.4</td>
</tr>
<tr>
<td>&gt;35</td>
<td>—</td>
<td>—</td>
<td>&gt;1645 / 27.4</td>
</tr>
</tbody>
</table>

**Conclusions**

The $LC_{50}$ by which to describe the toxic potency of fire smoke is employed to assess the degree of threat to human life. The relationship between $LC_{50}$ and mole fraction of wood-volatile is established. Since the mole fraction varies with time, a new approach is proposed to calculate the tenability in dynamic system. For exit staircases attached by fire, the formulation can be used to predict the tenability (measured in either seconds or minutes) condition for different storey heights.

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**References**


Heat Transfer Analysis for Insulated Steel Structure Exposed to Standard Fire

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Introduction

Temperature prediction of insulated steel structure exposed to fire, in general, involves 2D transient heat transfer analysis. In practice, the 1D characteristic heat transfer model, based on the lumped capacitance concept is commonly used. It provides a closed form formulation in steel temperature response when subjected to standard fire. Numerical iterations are involved if nonlinear material properties are assumed in the analysis, as outlined by Wickstrom [1] (1985) and later incorporated in Eurocode 3, part 1-2 [2].

The objective of this paper is to assess different schemes for evaluating temperature response of contour-insulated steel structure exposed to fire. Temperature-dependent thermal properties e.g., thermal conductivities, were assumed for insulation materials in both FEA and EC3 predictions, whereas the closed form solutions assume constant thermal conductivities of insulation for given thickness. Specimens included European steel I-sections, CHS and SHS with four surfaces exposed to standard fire and contour-insulated with intumescent Char 21 fire retardant paint. Test specimens and procedures were designed according to prENV 13381-4: 2001, with standard deviation controlled within tolerant range. The computational models of contour-insulated open and closed profile steel sections subjected to ISO 834 standard fire curve are shown in Figure. 1.

Analytical solution for 1D heat transfer model

The analytical solution for temperature response of insulated steel structure exposed to fire presented here is based on the 1D characteristic heat transfer model. The governing differential equation for conductive heat transfer in 1D Cartesian coordinates system is obtained

$$\frac{k}{\rho c} \frac{\partial^2 T(x,t)}{\partial x^2} = \frac{\partial T(x,t)}{\partial t}$$

where \(k\), \(\rho\) and \(c\) denote the heat conductivity, density and heat capacity of the conduction medium, respectively. \(T(x,t)\) denotes the temperature as a function of coordinates and time, \(x\) and \(t\) are the spatial and temporal independent variables.

Insulated steel structures in this paper are tested and modeled under standard ISO 834 fire curve. The standard heating curve is approximated by a finite sum of exponential terms as,

$$T_g(t) = T_0 + \sum_{j=1}^{3} B_j \exp(-\beta_j t)$$

where \(T_g(t)\) is the fire gas temperature, \(T_0\) is the initial temperature, constants \(B_j\) and \(\beta_j\) are as given in Table 1.

Table 1. Constants in the exponential expression of the ISO 834 fire curve

<table>
<thead>
<tr>
<th>(j)</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>(B_j) (°C)</td>
<td>1325</td>
<td>-430</td>
<td>-270</td>
<td>-625</td>
</tr>
<tr>
<td>(\beta_j) (h⁻¹)</td>
<td>0</td>
<td>0.2</td>
<td>1.7</td>
<td>19</td>
</tr>
</tbody>
</table>

The closed form solution of temperature distribution in the insulated steel structure subjected to ISO fire for \(t > \bar{t}\), is derived by Wickstrom as

$$T_i(t) = \bar{T} + \sum_{j=1}^{3} \frac{B_j}{\beta_j} \left[ \exp(-\beta_j(t-\bar{t})) - \exp(\bar{t} - \tau) \right]$$

where \(\tau\) is the steel column time constant, given as

$$\tau = \frac{R_i (Q_i + Q_s / 3)}{A} = \frac{c_i \rho_i d_i}{H_p / A_i} k_s \left(1 + \frac{\mu}{3}\right)$$

where the subscripts \(i\) and \(s\) respectively denote the properties of intumescent paint and steel; \(R_i\) and \(d_i\) are respectively the thermal resistance and thickness of insulation; \(Q_i\) and \(Q_s\) are respectively the lumped heat capacity of steel and insulation; \(A\) is the lumped area of 1D heat transfer model, \(H_p\) and \(A_i\) are respectively the length of perimeter and area of steel sections and \(H_p / A_i\) is the steel section factors, and \(\mu\) is the dimensionless ratio of lumped heat capacitance of insulation to that of steel, which is given as

$$\mu = \frac{Q_i}{Q_s} = \frac{c_i \rho_i (H_p / A_i)}{c_s \rho_s} d_i$$

and \(\bar{t}\) is the time shift of steel response, determined by

$$\bar{t} = \frac{\mu \bar{T}}{8}$$

For a general nonlinear case of insulated steel structure where the material properties vary with temperature, numerical calculations are required. An approximation expression for the stepwise increase of steel temperature \(\Delta T_s\) during a time interval \(\Delta t\) is given by Eurocode 3, as

$$\Delta T_s = \frac{T_g - T_i}{\tau} - \left[ \exp(\mu/10) - 1 \right] \Delta T_g$$
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Wickstrom’s method and Eurocode 3 both assume uniform temperature in the steel section when subjected to standard fire. The results show that the concept of “lumped capacitance method” works well. Lumped capacitance method by solving a 2D heat transfer problem using a 1D characteristic model thus simplifies the problem.

Conclusions

An assessment has been made on three different schemes for calculating the temperature of insulated steel structure subjected to ISO fire, namely, the closed form solution (Wickstrom’s method), Eurocode 3 prediction and FEA. It can therefore be concluded that application of numerical calculations accounting for material nonlinearity, i.e. EC3 and FEA, yield very good results in the estimation of steel temperature when compared with experimental results. For simplicity and design purpose, Wickstrom’s method is recommended as it predicts the steel temperature accurately above 400°C, i.e. the critical temperature for insulated steel structure design under standard fire.

References